SCHOOL OF
CIVIL ENGINEERING
INDIANA
DEPARTMENT OF HIGHWAYS

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Final Report

FHWA/IN/JHRP-92/20

Laboratory Evaluation of

Prefabricated Highway Edge Drains

Ahmed Mohamed Elsharief

PURDUE UNIVERSITY

Joint Highway Research Project

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Final Report
LABORATORY EVALUATION OF PREFABRICATED HIGHWAY EDGE DRAINS

To: Dr. Vincent Drnevich, Director October 5, 1992
Joint Highway Research Project Project No. HPR-2038 (29)
File No. 6-9-11

From: C. W. Lovell
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Subj: Laboratory Evaluation of Prefabricated Drains

Attached is the final report on the subject study. The report has been prepared by Mr. Ahmed Elsharief, Graduate Research Assistant, under my direction.

This study evaluated prefabricated highway edge in order to to establish guidelines for their proper selection. Two primary evaluations received attention: the first was the effect of installation and in-service loading on the hydraulic and strength capacities of the drain; and the second was the performance of the geotextile filter. The findings of this study were based on field investigation, laboratory testing and theoretical modeling. Generic guidelines were given, for the selection of both the core and geotextile.

The finding of this study will help INDOT to establish specifications for prefabricated highway edge drains. The developed retention criterion for nonwoven geotexties can be useful for other applications, e.g., selection of geotextiles filters for retaining wall sheet drains and for wick drains.

This report is submitted for acceptance in fullfilment of the referenced study.

Respectfully Submitted

CW Lovell

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cc:

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Final Report

LABORATORY EVALUATION OF PREFABRICATED
HIGHWAY EDGE DRAINS

by

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Joint Highway Research Project

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and the
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The contents of this report reflect the views of the author who is responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This report does not constitute a standard, specifications, or regulations.

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16. Abstract <p>This study evaluated prefabricated highway edge drains to improve the existing practice and to set guidelines for their proper selection. Two primary evaluations received attention: the first was the effect of installation and in service loading on the hydraulic and strength capacities of a prefabricated highway edge drain; and the second was the performance of the geotextile filter.</p> <p>A limited field evaluation of prefabricated highway drains indicated structural failure of the core and an inefficient geotextile. Based on, the field study, laboratory testing and theoretical modeling, guidelines were given for the strength and hydraulic requirements of the drain. A retention criterion was given for the geotextile filter. The criterion was based on reliability theory and accounted for the overall pore size distribution, porosity and thickness of the geotextile. Field geotextile samples were evaluated for their long term performance using image analysis. The evaluation indicated only partial clogging of geotextiles in service for 4 to 7 years.</p>			
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IMPLEMENTATION REPORT

Laboratory Evaluation of Prefabricated Highway Edge Drains

Introduction

The limited field investigation of prefabricated edge drains installed in Indiana, carried by this investigator, pointed out numerous problems which are related to the core and the wrapped geotextile. Similar problems were reported by investigators from other states (refer to Chapter 2 and 5). An intensive investigation and research was conducted to identify and understand the causes of the problems, and to suggest guidelines and remedial measures to minimize their effects. This part of the report will give the proposed guidelines for the selection of prefabricated highway edge drains. The guidelines are summarized in tabular format. More details are available in the previous chapters. These guidelines are generic, i.e., not intended for any specific product and are recommended for use in the state of Indiana.

Guidelines for the selection of prefabricated highway edge drains

Drainage Core			
hydraulic requirements	recommended values	recommended test methods	comments
	minimum inflow capacities of 15-18 gal/min/ft	ASTM D4716-87	the drain should be installed with a longitudinal slope of 2% for 15 gal/min/foot and 1% for 18 gal/min/ft
strength requirements	minimum normal (90°) compressive strength of 8100 psf and minimum inclined (80°) compressive strength of 7500 psf	GRI ¹ test method is recommended	*Values are based on the resistance to both compaction induced stresses and in-situ stresses. ** an equipment similar to the one shown on Figure 2.6 may be used for inclined loading
The Geotextile			
1. Strength and Durability Requirements			
puncture strength	recommended values	recommended test methods	comments
	$P_d = 39.5(A_t - A_s)$ <p>P_d is the required puncture strength; A_t is the geotextile area between centers of adjacent protrusions; and A_s is the area of a protrusion head</p>	ASTM D4833-88	i) The ASTM test closely simulates field conditions ii) The equation does not apply to cores which do not have protrusions
elongation	min. 15%	TF ² 25 #1	After AASHTO-ARBTA

¹Test method GC-4, "Standard Method for Compression Behavior of Prefabricated Edge Drains and Sheet Drains". It is a normal compressive loading test which was developed at the Geosynthetic Research Institute, Drexel University, Philadelphia, PA, especially for prefabricated highway edge drains and sheet drains

²TF stands for task force

continued.....

burst strength	min. 130 lbs	TF 25 #1	After AASHTO-ARBTA
trapezoidal tear	min. 35 lbs	ASTM D4533	After AASHTO-ARBTA
ultra violet degradation	70% strength retained	ASTM D4355	After AAHTO-ARBTA
2. Filter Requirements			
retention criterion	recommended values	recommended test methods	comments
	$t = \frac{O_{98} \log(1-R)}{\log(n-np)}$ <p>t is the required thickness of geotextile; O_{98} is the maximum pore size of the geotextile and is obtained from PSD; n is porosity; and p is the cumulative percentage of pores smaller than the particle size to be retained; R is the reliability (n, p and R are in decimals)</p>	pore size distribution should be obtained using mercury intrusion porosimetry	<p>i) The criterion is applicable to nonwoven geotextiles</p> <p>ii) Stability of the adjacent soil should be checked³</p> <p>iii) R is 0.8 for general applications and is 0.9 for PFHEDs.</p> <p>iv) p=0 when retaining fine particles</p>
General Comments			
<p>1)Appropriate construction control and maintenance are a necessity for satisfactory and prolonged performance of PFHEDs</p> <p>2)Borescope evaluation is recommended for the detection of construction related problems</p> <p>3)Outlets should be marked and regularly cleaned</p>			

³refer to Chapter 5

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LIST OF ABBREVIATIONS

PFHED	Prefabricated Highway Edge Drain
FHWA	Federal Highway Administration
AASHTO	America Association of State Highway Transportation Officials
ASTM	American Society of Testing and Materials
DOT	Department of Transportation
AOS	Apparent Opening Size
FOS	Filtration Opening Size
EOS	Equivalent Opening Size
GRI	Geosynthetic Research Institute
FIH	Franzius Institute of Hanover
CFG	French Committee of Geotextiles and Geomembranes
IRIGM	Institute of Grenoble
EPM	Ecole Polytechnique of Montreal in Canada
PSD	Pore Size Distribution
MIP	Mercury Intrusion Porosimetry
Q570	Quantiment
SEM	Scanning Electron Microscopy
SSA	Specific Surface Area
1-d	One Dimensional
log	Logarithm to the base 10

TECHNICAL SUMMARY

Introduction

Prefabricated edge drains have been recently used in lieu of conventional retrofit edge drains, mainly for their economy. Several products specially designed for highway drainage are now available in the market. An efficient prefabricated highway edge drain (PFHED) should have components satisfying three requirements: 1) the core should have drainage capacity adequate to quickly remove water from the pavement section; 2) the core and fabric should have adequate strength to resist the imposed stresses during installation and in service; and 3) the fabric should be able to retain the adjacent soil and allow minimum piping of fines into the core.

The general objective of this research project was to determine the properties of PFHEDs and to develop effective and acceptable criteria for their selection. To attain these objectives a comprehensive study which included field investigation, laboratory testing and theoretical modeling was carried out.

Results

- 1) The core should have a minimum flow capacity of 15-18 gallons/minute/foot (by ASTM-D4716-87) for longitudinal slopes of 2% and 1%, respectively.
- 2) The core should have a minimum normal compressive stress of 8100 psf and an inclined compressive strength (at 80°) of 7500 psf. The direct compressive stress may be obtained using the Geosynthetic Research Institute (GRI) compressive strength test, while the inclined test may be performed with an appropriate device.
- 3) Borescope evaluation is recommended during construction.
- 4) The overall pore size distribution (PSD) of the fabric filter

should be determined. Mercury intrusion porosimetry (MIP) should be used as a standard method for the determination of PSD of nonwoven geotextiles. Suggestions for the test procedure are given in the complete report.

- 5) A probabilistic model, based on the PSD, thickness and porosity (n) of the geotextile is given. The model predicts the reliability (R) of a geotextile in retaining a certain soil. The thickness of the geotextile needed for a specific reliability can be estimated. The following equation can be used

$$thickness = \frac{O_{98} \log(1-R)}{\log(n-np)}$$

Where (p) is the cumulative percentage of geotextile pores smaller than the the size of particle to be retained (in decimals). A target reliability of 80% (0.80) for granular base and subbase soils and 90% (0.90) for silty soils (p=0) was suggested for highway subdrainage.

- 6) There is a potential for geotextile clogging when fine sand particles find their way to the geotextile.
- 7) The morphological evaluation of geotextiles obtained from the field indicated no clogging of the geotextile. The clogging level was found to be independent of the geotextile age. Inner caking can be a potential threat to the long term performance of the fabric if considerable amount of fines is carried by water in the drain.

Conclusions

This study evaluated PFHEDs in order to improve the existing practice in Indiana and to set guidelines for their proper selection. The results given above are a brief summary of the research findings. Details on the research approach, the field investigation, structural properties of nonwoven fabrics, and the factors related to the performance of geotextiles are given in the report.

CHAPTER 1: INTRODUCTION

Background

The importance of drainage for highways have been emphasized by many researchers and practitioners (e.g., Cedergren, 1980; and Moulton, 1989). The AASHTO Guide for Design of Pavement Structures (1986) includes drainage as an essential element of pavement design. Conventional (aggregate and pipe) retrofit edge drains have been widely used in highway rehabilitation programs to help remove water from the pavement. Recently, prefabricated drains made of synthetic materials are used as viable alternatives to conventional edge drains, mainly due to their economy. A prefabricated drain is composed of a drainage core to convey water and a wrapped geotextile which is used as a filter. For satisfactory performance a prefabricated highway edge drain (PFHED) should have a drainage capacity sufficient to remove water trapped inside the pavement. The geotextile filter should be able to retain the base material and allow minimum piping of fines into the drainage core.

In Indiana, prefabricated drains were first used in highways in 1984 on an experimental basis. Since then, hundreds of miles of PFHEDs have been installed. There are several reasons which make the investigation of the performance of PFHEDs a necessity at this stage:

- 1) The drains have been recently introduced and their

introduction was not preceded by any serious study.

2) Different kinds of PFHEDs have been introduced by different manufacturers, with each promoting particular products, whereas, there are no standard guidelines or measures for the proper selection of these products.

3) Field experience indicated several problems related to the structural integrity of the core and the filter performance of the geotextile (e.g., Fleckenstein and Allen, 1991; and Baldwin et al, 1991).

Research Objectives

The general objectives of this research project were to investigate the various factors which affect the performance of PFHEDs, determine their properties, and develop acceptable and effective criterion for their selection. The principal objectives of this research were:

- 1) specifying laboratory tests for the determination of the mechanical and hydraulic properties of the drain;
- 2) developing guideline specifications for the hydraulic and strength requirements of PFHEDs for highways in Indiana;
- 3) gaining a better understanding of the structural properties of nonwoven geotextiles and assessing the laboratory test methods used for the determination of the pore size parameters of these materials;
- 4) developing a more efficient filter criterion for the selection of geotextile filters based on the overall pore size distribution of the geotextile; and

5) evaluating the long term performance of the geotextile filters.

Research Approach

In order to accomplish the above objectives, the following tasks were accomplished:

- 1) Literature review; an intensive literature review was made. The review covered the properties, performance and specifications of PFHEDs. A recent study by Koerner and Hwu (1989) involved intensive laboratory testing of typical PFHEDS. The results of this investigation were used for the estimation of the hydraulic and strength properties of the drains. An intensive review on the filtration characteristics of nonwoven fabrics was also done.
- 2) Field study; a limited field investigation program for the evaluation of PFHEDs was carried out. Drains installed 8, 6, and 4 years ago were exhumed and samples of the drains were cut and brought to the laboratory for further studies.
- 3) The drainage core; the drainage core was investigated for its hydraulic capacity and structural integrity. The flow requirements "demand" of the drain were obtained based on field data and empirical estimates of the amount of flow. The low capacity of the drain was obtained based on laboratory simulations. Specifications for the hydraulic requirements of the drain were developed on the basis of capacity versus demand for conditions in Indiana. The stresses to which PFHED components might be subjected during installation,

"demand", and in-service were estimated based on simple modeling. Laboratory test methods were also suggested to determine the strength "capacity" of the drain under different loading conditions. Drain specification were also developed based on capacity versus demand.

4. The geotextile filter; an intensive investigation of the filtration performance of non-woven geotextiles was made. The investigation involved:

- a) Measurements of the pore parameters of non-woven geotextiles by the mercury intrusion porosimetry and image analysis methods.
- b) Development of a probabilistic model to predict the retention ability of non-woven geotextiles. The model was based on the overall pore size distribution, porosity and thickness of the geotextile. It considered the geotextile compressibility and the suffusion characteristics of the soil.
- c) Assessment of the long term performance of field samples of geotextile by image analysis.

Contents of the Report

This report is comprised of 7 Chapters including this chapter. The second chapter, Chapter 2, covers the review of the literature. The review is divided into two major parts; in the first part the topics related to the drain as a whole are given, while, the second part reviews the topics related to the geotextile fabric.

Chapter 3 contains the results of the field investigation and

the design requirements for PFHEDs.

Chapter 4 contains the results and a discussion on the results of the tests performed to measure the pore parameters of 14 nonwoven fabrics obtained from different manufacturers, i.e, mercury intrusion porosimetry and image analysis. A comparison of the results with the current techniques used to determine the pore size parameters is given.

The probabilistic model developed to predict the retention ability of nonwoven fabrics is given in the Chapter 5. The Chapter contains test results used to validate the model, in addition to a proposed retention criterion.

The results of the analysis of a set of laboratory data on long term performance of geotextiles are given in Chapter 6. The Chapter also includes the results of a morphological evaluation of geotextiles from the field.

Chapter 7 contains the conclusions and recommendations of this study.

CHAPTER 2: LITERATURE REVIEW
PART (1): PREFABRICATED EDGE DRAINS

Importance of Drainage for Highways

Among the reasons cited for pavement failures, inadequate base drainage has been identified as a major contributor. Cedergren (1988) pointed out that the adverse actions caused by lack of drainage are: erosion and pumping of the base coarse and/or subgrade; stripping of asphalt coatings from bituminous treated bases and subbases; and over-stressing of weakened subgrades. The mere presence of abundant water causes or accelerates numerous non-load bearing actions such as D-cracking, blow ups, frost action, expansion and shrinkage cracking, etc.

To reduce moisture effects on the pavement two major approaches may be followed: 1) all pavement joints and cracks should be sealed to reduce infiltration into the pavement; 2) a drainage system can be used to remove the infiltrating water as quickly as possible. Therefore, acknowledging the seriousness of inadequate drainage, many highway agencies are retrofitting drainage on existing pavements and/or including free draining bases on new and reconstructed pavements. The AASHTO Guide for Design of Pavement Structures (1986) includes drainage as an essential element of pavement design. Thus, the methods of drainage include: free draining bases, french drains, pipe and aggregate subdrains, and most recently prefabricated "geocomposite" edge drains.

This part of the review will cover different topics related to the properties and performance of prefabricated highway edge drains.

Prefabricated Edge Drains

A Prefabricated highway edge drain, PFHED, consists of a plastic core, usually rectangular shaped, wrapped by a geotextile which is usually non-woven. The plastic core serves as a drainage medium while the geotextile acts as a filter, retaining soil particles while allowing water to flow. Many different products, specially manufactured to be used in highway subdrainage, are available in the market. Figure 2.1 shows some of the designs more frequently used in highway subdrainage. The drain is placed immediately adjacent to the pavement/shoulder joint (Figure 2.2).

The primary advantage of PFHEDs, when compared to conventional drains are (Koerner and Hwu, 1991):

- 1) ease of placement; the trench in which the PFHED is placed is only 4-6 inches wide and the same excavated material is used for backfill. The placement process is easy and fast, i.e, up to 50 feet of drain can be placed per minute; and
- 2) low cost; due to their ease of placement and the type of material from which they are made, PFHEDs are found to cost only half that of conventional edge drains.

Performance of Prefabricated Edge Drains

There are different opinions as to the effectiveness of

Double Cusped
Perforated Core
Color - Black
Weight 150 gm/ft²

Conical Cusped
Perforated Base
Color - Yellow
Weight 181 gm/ft²

Oblong Corrugated
Pipe Section
Slotted Perforations
Color - Black
Weight 377 gm/ft²

High Profile columns
Perforated Base
Color - Black
Weight 223 gm/ft²

Figure 2.1. Structural profiles of the drainage core of typical PFHEDs (after Frobel, 1991)

Figure. 2.2. Placement of a typical PFHED

various remedial pavement drainage techniques, specially PFHEDs for the following reasons: 1) the drains were recently introduced (i.e., in the early 1980's), therefore, there is a lack of good documentation and experience as to their effectiveness, durability and performance; 2) there are many variables which contribute to pavement distress and it is not easy to separate the effect of these variables including the effect of the drains; 3) engineers are not familiar with the synthetic materials from which the drains are made; and 4) various kinds of drains are in the market and all are claimed by their manufacturers to be competitive.

Although vastly different in design and construction, the current products are intended, or supposed, to perform the same function, i.e., to successfully remove water from the pavement. However to perform the above function, the following requirements are necessary:

- 1) the drain should be strong enough to resist handling, installation and in-service stresses;
- 2) it should possess adequate flow capacity, exceeding the inflow to the drain; and
- 3) the geotextile filter should be able to pass the inflow while retaining the adjacent soil.

Recognizing the need to study the effectiveness of edge drains, the Federal Highway Administration (FHWA) initiated a special experimental project named "Experimental Project No. 12" to evaluate and document the effectiveness of retrofitting longitudinal edge drains in concrete pavements (Baumgardner and Mathis, 1989).

The study covered 10 states and considered both conventional drains and PFHEDs, with more emphasis on the former. The results of this investigation are not yet available for publication. Different states are also assessing the performance of their drains.

Different methods exist for evaluating the performance of PFHEDs such as: 1) borescoping by inserting a video camera from the shoulder down to the drain and visually inspecting the drain; 2) exhuming the drain and visually inspecting it; and 3) observing the amount and quality of the outflow.

Kentucky's Experience

Fleckentein and Allen, (1991) reported the results of investigations carried out to evaluate the performance of PFHEDs in Kentucky. Several sites were investigated after recognizing localized pumping where PFHEDs were installed. Three type of drains were used in the problem areas, type (A) (Hydraway by Monsanto Chemical Co.), type (B) (Advanedge by Advanced Drainage System, ADS) and type (C) (Akwadrain by International Construction Equipment Inc.). This study is comprehensive will be summarized in three parts. A summary of the type of problems encountered will be given in the first part, the causes of the problems in the second part and the recommended remedial measures in the third part.

Type of Problems

- 1) Compression of the inner part of the panel drain during compaction of the backfill. This was found to be mostly

critical for type (A) drains.

- 2) Damage to the flexible outlet pipes during construction.
- 3) Improper placement of outlet pipes.
- 4) Siltation of the core.

Causes of the problems

Most of the above problems occurred because of improper and deficient construction practice, basically compaction induced stresses, improper handling during construction, and improper installation of outlets. Compression and shearing forces tended to fold over the cuspatations specially for type (A). Heavy compaction wheels were found to cause more problems compared to vibratory compactors. Breaking and seating operations produced silt size particles which could be carried by water into the drain.

Remedial Measures

Because of the above mentioned problems, Kentucky changed its specifications for installation of PFHEDs. The new specifications require the PFHED to be installed on the shoulder side of the trench and clean coarse sand be used for backfilling (Figure 2.3). Compaction is done by flushing the trench with water. The sand will act as a fill material and also as a first line of defense against the passage of fines into the drain.

According to the new specifications, the outlet distances are reduced from an average of 700 feet to a maximum of 450 feet for grades over 2% and 200 feet for grades less than 2%. This is

Figure 2.3. New installation procedure for PFHEDs in Kentucky
(after Fleckenstein and Alllen, 1991)

recommended in order to increase the rate of removal of water from the pavement.

To insure that the integrity of the core has been maintained, it is recommended that the core should be borescoped after installation.

West Virginia's Experience

Baldwin, (1991) presented the results of an investigation of a pumping incident on a rehabilitated highway in West Virginia, in which PFHEDs were installed. The incident was described as "unusual sporadic staining on the pavement shoulder". Outflow was observed to be good although sediments were observed in the outwash.

Two types of drains were used, type (A) (Hydraway) and type (B) (ADS). All prefabricated edge drain installations were 1 foot away from the edge of the pavement and 4 inch inches below the subgrade/subbase interface. The investigation included borescoping of the problem areas, excavation of several test pits and an outlet evaluation.

The borescoping of type (B) sites revealed the following: 1) no crushing of the inner flow channel; 2) up to 3 inches of fines settled in the drain, in the bottom of the channel; 3) many of the slit openings of type (B) drain, especially in the bottom two rows, were blinded with sediment. Test pit excavations on sites having both kinds of edge drains revealed the following; 1) apparent deformation of type (A) cores; and 2) When there was breaking and seating operations, the pavement acted like a dam preventing the flow of water to the edge drain.

The following remedial measures were suggested:

- 1) the PFHED should be placed in direct contact with the pavement slab; and
- 2) outlets should be constructed of smooth walled rigid pipes to help assure proper outlet grading.

Laboratory Testing of PFHEDs

In order for a prefabricated edge drain to perform adequately, the components of the drain, i.e., the core and the wrapped geotextile should be able to remove the trapped water from the pavement, and should also be able to resist the construction and in-situ stresses. The durability requirements should also be satisfied, i.e., resistance to fungus and bacteria and to UV light during handling.

Laboratory tests for PFHED components may be grouped into hydraulic tests, strength tests and durability tests. Tests related to the core will be reviewed first.

Hydraulic Tests

The basic function of a PFHED is to transmit water efficiently throughout the life of the drain, therefore, techniques are needed to estimate the flow capacity of the drain.

Currently the flow capacity of PFHEDs is measured in the laboratory using procedures described in ASTM specification D4716-87 "Standard test method for constant head hydraulic transmissivity (in-plane flow) of geotextiles and geotextile related products".

According to these procedures, the flow capacity per unit width is determined by measuring the time required for a specific quantity of water to pass through the test specimen under specific normal stress and specific hydraulic gradient. A schematic of the transmissivity device is shown in Figure 2.4. It is important to note that the drain is placed horizontally during the test, while it is vertically installed in the highway system. Although the total flow area is the same for both cases, we observe a difference in flow pattern, i.e., the flow is closer to pipe flow in the laboratory and to open channel flow in the field.

Dempsey, (1989) used a 7.3 meter long channel which can handle a 6.1 meter long PFHED sample in order to simulate field conditions in the laboratory (Figure 2.5). The drain is sandwiched between one side of a plexiglass flow channel and a plywood plate so that the width is in a vertical plane. The flow capacity of the drain can be measured at different entrance head levels (i.e., 15, 30 and 6.1 cm) and different channel flow line slopes (i.e., 0, 1, 2, 3 and 4%). Dempsey's flow channel is the only one that closely simulates field conditions.

Since PFHEDs are basically designed to transmit water, it is important to know of any factors that adversely affect their transmissivity. Haj Ali et al, (1990) and Kraemer et al, (1984) observed that the installation or construction process and in-place loading of prefabricated drains, cause structural changes and substantially affect the discharge capacity of the drain. The expected structural modifications are: 1) compressibility of the drainage core; 2) bending of the core; 3) intrusion of the filter

Figure 2.4. Hydraulic Transmittivity testing device
(after Dempsey, 1989)

Figure 2.5. Dempsey's long channel flow device
(after Dempsey, 1989)

fabric into the core space; and 4) siltation of the drain. These modifications are controlled by the geometrical structure of the core, the flexibility, creep and filtration characteristics of the wrapped fabric, the type of the adjacent soil, the hydraulic conditions, and the nature and amount of external loads (in-situ) applied to the drain.

Strength Tests

A basic requirement for a PFHED is that its components, i.e., the core and the wrapped geotextile should be strong enough to sustain the imposed stresses during construction and through the lifetime of the drain. Neither AASHTO nor FHWA have sponsored special testing programs investigating this behavior for PFHEDs. The ASTM standards that deal indirectly with the strength requirements of PFHEDs are presented in Table 2.1. They can be grouped into tests related to the drainage core and tests related to the fabric. A review of the existing tests used to evaluate the strength of PFHED components follows.

Compression Testing of the Core

To date, stress testing on core structures used in highway edge drains has been limited to normal compressive loading. The manufacturers report the "crushing resistance" of the core for their products, using various methods and rigid plate sizes. According to Kraemer and Smith, (1986), the factors which adversely affect the results of compression tests and the field performance

Table 2.1. ASTM standards dealing with PFHEDs
(after Koerner and Hwu, 1989)

ASTM designation	purpose	comments
ASTM G-21	Fungus Resistance	G&C ¹
ASTM G-22	Bacteria Resistance	G&C
ASTM D-570	Water Absorption	G&C
ASTM D-638	Tensile Properties of Plastics	C
ASTM D-792	Specific Gravity by Displacement	C
ASTM D-695	Compressive Strength	C
ASTM D-1248	Polyethylene Resin Extrusion	C
ASTM D-1505	Specific Gravity by Density Column	C
ASTM D-1621	Compressive Strength	C
ASTM D-1876	Peel Strength of Geotextile-to-Nodes and Geotextile to Core	C
ASTM D-2990	Creep Rupture of Plastics	C
ASTM D-3350	Polyethylene Resin Pipe and Fittings	C
ASTM D-4716	In-Plane Transmissivity Flow Rate	C
ASTM D-1876	Peel Strength of Geotextile-to-Geotextile	G&C
ASTM D-1910	Mass per Unit Area	G
ASTM D-3786	Burst Strength	G
ASTM D-3787	Puncture Strength	G
ASTM D-4355	UV Exposure and Accelerated Weathering	G
ASTM D-4491	Permeability (Permittivity)	G
ASTM D-4533	Trapezoidal Tear Strength	G
ASTM D-4632	Grab Tensile Strength and Elongation	G
ASTM D-4751	Apparent Opening Size	G

¹G stands for geotextiles; C stands for drainage core

are small sample size, eccentricity of loading, and presence of secondary yield phenomena. Eccentric or inclined loads are found to adversely affect the compressive strength of cores that are highly anisotropic.

Koerner and Hwu, (1989), feeling the need for a specific compressive strength test for highway PFHEDs, suggested a test method entitled "Test method GC-4, Compressive Behavior of Prefabricated Edge Drains and Sheet Drains". The test is a normal compressive loading test and was intended as an index test. It was anticipated that the results of the test will be used to evaluate product uniformity and stability as well as the impact of changes in product methodology. The core crushing strength, obtained from this test may be used for design purposes. The test is the most comprehensive test method available for edge and sheet drains.

Following a different approach, and in an effort to study the effect of loading eccentricity on core types in the laboratory, Frobels, (1991) used a special load frame device designed to make possible the application of an inclined load to the test sample. Figure 2.6 shows a conceptual drawing of the frame. The rotating base table in the figure can be locked into position at increments of 10 degrees to the normal. Failure loads are obtained for each inclination.

Durability Tests

The ASTM tests which address the durability of the core are ASTM G21 on Fungus Resistance and ASTM G-22 on Bacterial Resistance.

Figure 2.6. The eccentric load frame device (after frobel, 1991)

Specifications of PFHEDs for Highways

Koerner and Hwu, 1989 reported the status of specifications of the state transportation agencies for installation and selection of PFHEDs. Only New York and Florida were found to have formal specifications. Some states had only draft specifications. Most of the specifications use ASTM test methods. New York Department of Transportation (DOT) specifications contain a list of approved products, while the specifications of Florida DOT do not. The specifications cover detailed requirements for the drain, filter selection, installation and basis of payments.

Indiana has not yet developed specifications, and Hydraway is the only drain widely used in the state. The manufacturers specifications are followed for the installations of this type of drain.

PART (2): GEOTEXTILES

The following definition has been adopted for geotextiles by the American Society of Testing and Materials (ASTM) " a geotextile is any permeable textile material used with foundation, soil, rock or any other geotechnical engineering-related material as an integral part of a man-made project, structure or system".

Types of Geotextiles

The types of geotextiles that comply with the definition are knitted woven and non-woven fabrics which are produced by the textile industry. The woven geotextile compose of a set of parallel yarns systematically interfaced to form a planar structure. Non-woven fabrics are formed from filaments and or short fibers arranged in all directions in a loose web and bonded together mechanically "needle punched", chemically "resin bonded" or thermally "heat bonded" (ASTM D-123, 1985).

Most geotextiles are made from synthetic polymers such as polypropylene, polyester, polyethylene and polyamide (nylon). These polymers are known to have a high resistance to the natural environment, i.e., chemical and biological attacks, compared to natural fibers. Geotextiles are therefore identified by the kind of polymer(s), type of fibers or yarns, type of geotextile and mass per unit area (Giroud, 1987).

Geotextile Properties

The important properties for civil engineering applications may be grouped into: structural properties; mechanical properties; and hydraulic properties.

Structural Properties

The important properties which are measured by index tests are specific gravity, mass per unit area, thickness and pore opening sizes. The specific gravity and mass per unit area are direct evaluation of the solid constituent of the geotextile while the thickness and pore opening sizes relate to the fabric structure. The mass per unit area is an important property to manufacturers since it is directly related to cost. Currently, the thickness of the geotextile is more a descriptive property than a design oriented property (Koerner, 1990). The structural properties of geotextiles can be divided into macro and micro structural properties. The macro-structural properties are thickness and porosity, while the micro-structural properties include pore size distribution, fiber thickness, and fiber arrangements for non-woven.

Several techniques have been used to measure the structural properties, basically, the pore size distribution of non-woven geotextiles. These include theoretical developments, apparent opening size (ASTM D4751), effective opening size, D_w (suggested by Franzius Institute, Hanover University), the hydrodynamic sieving method (Rollin et al, 1990), image analysis, mercury intrusion

porosimetry (Prapaharan and Holtz, 1987). A review of the principles and limitations of the current methods which are widely used and are related to this research topic is given below.

Theoretical Developments

Theoretical developments were reported in the literature to determine the pore size distribution of non-woven geotextiles. Most of them were based on probability-based theories. Faure et al (1986)^a showed, based on previous developments by Matheron (1971), that the cumulative probability, $y(d)$, to have a pore opening less than d can be given by:

$$y(d) = P(O < d) = 1 - \left[1 + \frac{2(1-n)}{\pi} \frac{T_e}{d_f^2} \cdot d \right]^{\frac{2t}{T_e}} \exp \left[-\frac{4(1-n)}{\pi} \frac{t}{d_f^2} d \right] \quad (2.1)$$

Where:

n is porosity

t is thickness

d_f is the fiber thickness

T_e is thickness of an elementary plane of the fabric

The parameters needed for the above equation are n , t , d_f and T_e . Faure et al, (1986)^b suggested ($T_e = d_f$) be used. Comparisons with pore sizes obtained by dry sieving showed the calculated pore sizes to be smaller than the measured.

Rollin et al, (1982) and Masounave et al, (1980) used a semi-Markovian process to obtain the dimensions of voids between fibers of a cross section of a thick non-woven geotextile. It was assumed

that, in a cross section of a typical fabric, the fibers are circular and randomly distributed following Poisson's law, (Figure 2.7). The dimensions of voids between fibers was obtained using probability theory. The probability of a circle being of a diameter $x < X$ is:

$$F_x(x) = P(x \leq X) = 1 - \exp\left(-\frac{1}{n} \left(\frac{\tau \pi x^2}{4} + \frac{2\lambda x}{2} \right)\right) \quad (2.2)$$

Where:

n is porosity

τ is fiber density

2λ is the fiber perimeter

The probability of a circle's diameter being between a value x and $x+d_x$ is given by:

$$f_x(x) = -\frac{1}{n} \left(\frac{\pi \tau x^2}{4} + \frac{2\lambda}{2} \right) \exp\left(-\frac{1}{n} \left(\frac{\pi \tau x^2}{4} + \frac{2\lambda x}{2} \right)\right) \quad (2.3)$$

For further simplification Rollin et al, (1980) assumed that $q=0$ and $2\lambda = 0$. Therefore Equation (2.2) becomes:

$$f_x(x) = -\frac{\pi \tau}{2} x \exp\left(-\frac{\pi \tau x^2}{4}\right) \quad (2.4)$$

The only parameter that is needed is τ , the fiber density. The most probable pore diameter d_p , the mean pore diameter $\bar{d}$ and the maximum pore diameter $d_{\max}$ were obtained from Equation (2.3) and are given as:

Figure 2.7. Schematic cross section of a fabric
(after Masounave et al, 1980)

$$d_p = 2 \left(\frac{1}{2\pi\tau} \right)^{0.5} - d_f^- \quad (2.5)$$

$$d^- = \left(\frac{1}{\tau} \right)^{0.5} - d_f^- \quad (2.6)$$

$$d_{\max} = 2.1 \left(\frac{1}{\tau} \right)^{0.5} - d_f^- \quad (2.7)$$

A pore size histogram of a fabric can be determined if the fiber density is known. Rollin et al, (1990) used an image analyzer to determine τ .

The Apparent Opening Size

The first test to characterize the opening size of geotextiles was developed by Calhoun at the Corps of Engineers Waterway Experiment Station in the late sixties (Calhoun, 1972). The test was developed to characterize the soil particle retention ability by sieving rounded sand particles of specified sizes through the fabric, to determine the fraction of particle sizes for which 5% or less by weight passed through the fabric. The equivalent opening size, EOS, is defined as "the retained on" U.S. standard sieve number of this size fraction. ASTM has approved a similar test procedure, ASTM D4751, to evaluate the apparent opening size, AOS, which is a modified EOS test in which glass beads are used instead of rounded sand. The term "apparent" was introduced because the results do not quantify equivalence between a fabric and U.S. standard sieve or another fabric. The apparent opening size, AOS, has been widely used to represent the structural properties of geotextiles in their filter criteria.

It is important to note that the AOS values do not accurately define the pore structure of the fabric. AOS shows no relationship to fabric soil system performance during filtration (Carrol, 1983). Several factors cause the AOS test to be rated as a poor index test when characterizing pores (Koerner, 1990; and Carrol, 1983; 1987). These factors are:

- 1) build up of static electricity on the glass beads;
- 2) trapping of particles in the geotextile;
- 3) difficulties in testing thick fabrics; and
- 4) ignoring the distribution of pores with sizes less than AOS.

Mercury Intrusion Porosimetry

The idea of forcing a non-wetting fluid, mercury, into the pores of a material in order to determine its pore structure was first introduced by Washburn, (1921). The idea was based upon the principle that the surface tension of a non-wetting fluid will oppose the entry of the liquid into the small pores of a material. This opposition could be overcome by an external pressure that is inversely related to the pore diameter. For cylindrical pores the apparent pore diameter d is given by:

$$d = \frac{-4 T \cos(\theta)}{p} \quad (2.8)$$

Where:

p is the applied absolute pressure

T is surface tension of mercury

θ is contact angle between mercury and the pore wall

d is the size of pore which can be intruded at pressure P , called the apparent pore diameter.

The major assumptions and sources of error of this method are discussed in detail by Ritter and Drake, (1945), Grace, (1956); Winslow, (1984); Garcia-Bengochea, (1978) and others. The important assumptions inherent in Equation (2.8) are: 1) the pore cross sections are circular; and 2) the contact angle and surface tension remain constant during measurements on a single sample. Deviations from the assumed circular cross section is however the main source of error (Ritter and Drake, 1945). For non-circular pores the true pore size is considered to be the radius of the largest possible inscribed circle, and the constant 4 in Equation (2.8) can vary between 2 and 6 for extremes of non-circular pore shape, and packing, (Grace, 1956). Non-rigidity of the pore system under the applied loads may conceivably lead to compression of the sample during measurement and to distortion of the pore size structure. However, distortion is mainly a factor for pore radii corresponding to an absolute pressure greater than atmospheric pressure.

Two methods have been used to graphically present the pore size distribution: the differential distribution curve and the cumulative distribution curve. The latter displays the fraction of pore space greater or smaller than a given size. The differential distribution function is derived by Ritter and Drake, (1945) to be:

$$D(r) = 2 \frac{p}{d} \frac{dV}{V_o \delta p} \quad (2.9)$$

where dV is the volume of pore between (d) and $(d+\delta d)$ and V_0 is the maximum intrusion. This differential distribution function is suitable when the range of pore sizes extends over a narrow band. Another form of the differential distribution curve suitable when the range of pore diameters is very wide is illustrated in Garcia-Bengochea, (1978). Prapaharan and Holtz, (1987) used mercury intrusion porosimetry to determine the pore size distribution of non-woven geotextiles. They found the method to be simple, consistent and rapid. An ASTM standard was developed for the determination of pore size distribution, by mercury intrusion porosimetry, of soils and rocks (ASTM D-4404).

The Filtration Opening Size

This is an indirect method of measuring the pore size characteristic of geotextiles. It was developed in France and is based on hydrodynamic filtration. It basically consists of installing a geotextile specimen at the bottom of a cylinder. A selected granular testing material is placed on the specimen surface. The cylinder is forced in and out of the water using a mechanical, electrical or pneumatic device, Figure 2.8. The motion causes the passage of particles through the specimen. The filtration opening size, FOS, is the d_{95} of the tested material fraction passing through the geotextile. The method is most suitable for non-woven needle-punched geotextile (Rollin et al, 1990). The FOS measured by the hydrodynamic sieving method is

Figure 2.8. A schematic diagram of the FOS apparatus
(after Rigo et al, 1990)

affected by three categories of parameters: the test conditions; particle properties and soil distribution; and the geotextile structure.

Rollin et al, (1990), after investigating the above parameters, emphasized the need for standardizing the test conditions. Considerable variation is obtained when different particle types were used to measure the FOS, therefore, they recommended the use of glass beads to determine FOS because of the excellent re-productibility of the results. Graded sand or glass beads are used for the test and FOS was found to be sensitive to gradation.

Rigo et al, 1990 reported a strong relationship between FOS and mass per unit area, Figure 2.9. A relationship was also found with thickness for geotextile with the same kind of manufacturing process Figure 2.10. However, no significant relationship was found between FOS and porosity.

Comments

From the above review on the current methods used to determine the structural properties of geotextiles, it is clear that special emphasis was given to the pore size distribution of the geotextile. However, the current methods (AOS and FOS) to determine the pore size distribution of geotextiles, especially the non-woven, are approximate and have inherent discrepancies. The sieving techniques have the shortcoming of specifying a single pore size to characterize all the structural properties (AOS and FOS). The mercury intrusion porosimetry is probably most reliable and covers

Figure 2.9. The variation of FOS with mass per unit area
(after Rigo et al 1990)

Figure 2.10. Variation of FOS with T/d_f (after Rigo et al, 1990)

a wide range of pores.

Hydraulic Properties of Geotextiles

Liquids and gases can seep through a fabric either perpendicular to the plane of the fabric or parallel to it. When the flow is perpendicular to the fabric a conductivity measure can be either the coefficient of permeability or the permittivity. Because we are dealing with relatively thin materials the hydraulic gradient is not easily defined and the flow regime is not laminar, therefore, the permittivity (the coefficient of permeability divided by the thickness) is used. Permittivity is defined as "the volumetric flow rate of water per unit cross sectional area per unit head under laminar flow conditions in the normal direction through the geotextile" (Carrol, 1987). There is an ASTM standard to measure the permittivity of geotextiles, ASTM D4491-85.

Sufficiently thick geotextiles can transport fluids within their planes. The amount of fluid conveyed under a given hydraulic head is proportional to the hydraulic transmissivity of the geotextile. The transmissivity is equal to the in-plane permeability multiplied by thickness, if the flow is laminar. The transmissivity is linearly related to the compressibility of the fabric.

Mechanical Properties of Geotextiles

The mechanical properties include the tensile behavior, compressive behavior, resistance to concentrated stresses and

mechanical interaction with soils. There are well established standard tests to evaluate such behaviors, e.g., tests to evaluate the tensile, grab, tear, puncture, impact and bursting resistances (Koerner, 1990)

Geotextile Functions and Applications

The important functions of geotextiles are summarized in Figure 2.11 and are; fluid transmission, separation, protection and tensioned membrane. The purposes for which these functions are utilized are self explanatory in the Figure. However it is interesting to note that geotextiles usually perform more than one function at a time. Figure 2.12 summarizes the important geotextile applications. This review only focuses on the filtration function of non-woven geotextiles.

The Filtration Function of Nonwoven Geotextiles

Synthetic fabric membranes first came into use as an alternative to granular filters in the late fifties (Hoare, 1982). This use started to gain acceptance when high strength fabrics, developed specially for civil engineering works entered into the market. Currently geotextiles are widely used as drainage media for economy, and ease of placement in the field. However, potential disadvantages for fabrics are: they require careful handling during installation; they are affected by ultra-violet light; and their long term performance has not yet been evaluated.

Two conflicting requirements are needed to assure an optimum

Figure 2.11. Examples of geotextile functions
(after Giroud, 1987)

Figure 2.12. Examples of geotextile applications
(after Giroud, 1987)

performance of a geotextile filter: 1) it should have an adequate retention ability, or pores fine enough to retain the erodible soil; and 2) it should maintain adequate permeability to allow free flow of water out of the surrounding soil into the drain, or it must have pores open enough to allow free flow of water.

Filtration Mechanisms

An acceptable mechanism for geotextile filtration must include: the structural, and mechanical properties of the fabric; the characteristics of the soil to be protected, i.e, grain size distribution and stability; external stresses and strains; and the hydraulic conditions prevailing, i.e, laminar versus turbulent and reversing flow.

The mechanisms taking place during filtration have been described by many investigators (e.g., Hoare, 1982; Lawson, 1982; Rosen and Marks, 1985; and Rollin and Lombard, 1988). There is some kind of agreement, among them, about the existence of two filtration mechanisms when a geotextile and soil interact, mainly, a bridging mechanism and a vault mechanism.

When a geotextile is placed adjacent to a soil and water is allowed to flow, a complex interaction takes place between the two systems. Initially the soil particles adjacent to the geotextile that are smaller in size than the pores in the geotextile migrate into and through the geotextile. The soil particles larger than the pores of the geotextile rearrange and form a bridging network against the upstream surface of the geotextile. As water passes, fines get trapped in the bridging network and a condition takes

place in which larger particles stop smaller ones which consequently stop even finer ones until equilibrium is reached. The filtration mechanism could be well illustrated by Figure 2.13. This mechanism is favored in well graded soils with particles coarse enough to be stopped by the geotextile. A similar mechanism was observed for broadly graded soil (Lafleur et al, 1986). Thus, in such a mechanism, the geotextile does not filter the soil by itself, but rather it acts as a catalyst in promoting a stable soil filter from in-situ parent soils.

In some soils, such as cohesionless soils with appreciable clay content, the geotextile can properly be selected to favor vault formation, where particles adjacent to the geotextile rearrange themselves as they migrate towards the filter interface to form vaults Figure 2.14 (Mlynarek et al, 1991; Rollin and Lombard, 1988). Upon formation of the vault, the geotextile stops particles smaller in diameter than the modified opening size of the filter. The resulting network is believed to result from electrical and adsorption forces between organic lubricant on the fibers and the soil fabric and between the particles themselves. Certain factors accelerate the formation of vault network. These factors are low permeability and porosity of the geotextile, large thickness and surface characteristics that offer free fibers contact with the soil (Rollin and Lombard, 1988).

The movement of soil particles adjacent to the geotextile filter results in three conditions which affect the system performance. These conditions are:

- 1) Piping; which is the migration of fines through the

Figure 2.13. Schematic of natural filter with self filtration mechanism (Rollin and Lombard, 1988)

Figure 2.14. Cross section of natural filter with vault network formation (after Rollin and Lombard, 1988)

geotextile. This is undesirable since excessive loss of fines may result in instability of the adjacent soil, create preferential flow paths which may lead to the formation of large voids in the soil or may result in affecting the other components of the drainage system.

2) Clogging; small soil particles become trapped physically or electro-statically within the geotextile structure. This reduces geotextile flow rate and may cause system failure.

3) Blinding; fine soil particles depositing on the geotextile surface blocking the voids and impeding flow through the geotextile. The above conditions may occur separately or simultaneously in a typical drainage system.

Filter Criteria

For a successful use in filtration and drainage applications a geotextile filter must satisfy the following requirements (Christopher and Holtz, 1985):

- 1) Should prevent piping of the protected soil, i.e, satisfy a retention criteria.
- 2) The geotextile must remain more permeable than the adjacent soil throughout the lifetime of the project, i.e, satisfy permeability criteria.
- 3) Must have sufficient strength (i.e., tensile, burst and puncture) and must retain these properties throughout the lifetime of the project.

Numerous factors have been found to affect the behavior of a

geotextile filter (Williams and Luettich, 1990). These factors may be categorized into three groups: 1) soil properties; 2) boundary conditions; and 3) geotextile properties. Table 2.2 summarizes the individual parameters associated with each of the three categories. Due to the large number of variables involved in the filtration process of soil geotextile/systems, the design of geotextile filters is a complex process. Efforts towards specifying filter criteria for geotextiles follow two main approaches. The first approach is a modification of the specifications used for granular filters, while the second approach uses a laboratory simulation procedure. Special criteria and testing techniques are suggested when hydrodynamic flow conditions prevail.

Granular Filter Approach

The first filter criteria for granular filters was developed by Terzaghi (Terzaghi and Peck, 1967). The criteria consist of retention criterion and permeability criterion and is demonstrated by the following inequalities.

$$\text{retention} \quad d_{15f} < 4d_{85s} \quad (2.10)$$

$$\text{permeability} \quad d_{15f} > d_{85s} \quad (2.11)$$

Where:

d_{15f} is the grain size diameter of the filter, greater than 15% of the filter mass

d_{85s} is the grain size of the base soil, greater than 85% of the mass of the base soil

Table 2.2. Factors affecting the performance of geotextile filters in drainage systems (Williams and Luettich, 1990)

soil properties	geotextile properties	boundary conditions
type	type	hydraulic gradient
grain size distribution	primary bonding method	flow velocity
hydraulic conductivity	secondary bonding method	direction of flow
soil structure	thickness	state of stress
soil fabric	mass/area	stress history
void ratio	compressibility	drainage media
plasticity index	porosity	
degree of saturation	pore size distribution	
	permittivity or permeability	
	flexural rigidity and tensile strength	
	denure	
	density of needle punching	

Various other geometrical formulations have been developed for granular filters (e.g., Wittmann, 1979; and Sherard et al, 1984). However, the rationale behind most of these criteria is that it is the pores of the granular filter layer which are the active component of the filter, because the pores allow water to pass , while inhibiting the movement of the adjacent soil. The only reason for specifying particle size grading for granular filters is to provide specific pore size in the filter layer to filter in-situ soil properly (Lawson, 1982). Both the filter and the adjacent soil, are represented by effective grain sizes, such as, d_{85} and d_{15} .

The approaches which specify geotextile filter criteria, based on the granular filter rationale (Schober and Teindel, 1979; Rosen and Marks, 1985; and others), assume that a geotextile having a characteristic pore size equal to the average pore size of an equivalent granular filter will give similar performance. This is similar to designing for a granular filter and then selecting a geotextile with an effective opening size equal to that of the granular filter. A major disadvantage of this approach is that it ignores the structural differences between the two filter media.

A general formulation of the existing filter criteria for geotextiles is given as:

$$O_x < \zeta d_x \quad (2.12)$$

Where O_x is a characteristic pore size of the geotextile and d_x is an indicative grain size of the soil. The indicative pore size of the geotextile was adopted to be equal to the AOS (Calhoun,

1972), O_{90} (Schober and Teindel, 1979) and O_{50} (Rankilior, 1978). The character ξ is a dimensionless coefficient, dependent on the geotextile structure and soil gradation.

Schober and Teindel, (1979) introduced the effect of soil gradation and presented criteria in terms of d_{50} . Here the dimensionless coefficient is a function of the uniformity coefficient of the soil to be filtered. The relationship is presented in Figure 2.15 in which η is a factor of safety. Giroud, (1982) presented a filter criteria based on AOS and takes into account both the density and uniformity coefficient of the soil, Figure 2.16. A list of the existing geotextile filter criteria, which are based on deductions from granular filter criteria, is given in Table 2.3.

Fischer et al, (1990) presented a filter criteria based on the granular filter approach, but considering the overall pore size distribution of the geotextile (see Table 2.4). The criteria provide for both retention and clogging, with no distinction necessary for woven and non-woven geotextiles. The researchers suggested that mercury intrusion porosimetry be used to obtain the pore size distribution of the fabric because of its simplicity. These criteria, although considering the overall pore size distribution of the geotextile, have the same inherent weakness as the other granular-filter based criteria, i.e., they neglect the basic structural differences between the filter media of granular soils and geotextiles. Neither the porosity nor the thickness of the geotextile is considered.

Figure 2.15. Geotextile filter criteria by Schober and Teindel (1979)

Figure 2.16. Retention criteria by Giroud, 1982

Table 2.3. Geotextile filter criteria, based on granular filter approach (from Fischer et al 1990)

Source	Criteria	Remarks
Calhoun (1972) (from Rankilor, 1981)	$O_{95}/d_{85} \leq 1.0$	Woven, soils with less than 50% < #200
	$O_{95} \leq 0.2$	Woven, slightly cohesive soils
Zitscher (1975) (from Rankilor, 1981)	$O_{50}/d_{50} \leq 1.7-2.7$	Woven, soils with $C_u \leq 2$, d_{50} between 0.1 and 0.3 mm.
	$O_{50}/d_{50} \leq 25-37$	nonwoven, cohesive soils
Ogink (1975)	$O_{90}/d_{90} \leq 1.0$	woven
	$O_{90}/d_{90} \leq 1.8$	non woven
Sweetland (1977)	$O_{15}/d_{85} \leq 1$	nonwoven, soils with $C_u=1.5$
	$O_{15}/d_{15} \leq 1$	non-woven, soils with $C_u=4$
ICI Fibers (1978) (from Rankilor 1981)	$O_{15}/d_{85} \leq 1$	Nonwovens, soil with $0.02 \leq d_{85} \leq 0.25$
	$O_{15}/d_{15} \geq 1$	nonwovens, soils with $d_{85} > 0.25$ mm
Schober and Teindl (1979) no factor of safety	$O_{90}/d_{50} \leq 2.5-4.5$	woven and thin non-woven, dependent on C_u , silt and sand
	$O_{90}/d_{50} \leq 4.5-7.5$	thick nonwovens, dependent on C_u , silt and sand
Miller Ho and Turnbull (1980)	$O_{50}/d_{85} \leq 1.0$ $O_{50}/d_{15} \leq 1.0$	woven and nonwovens
Giroud (1982)	$O_{95}/d_{50} \leq 1-6$ $K_f \geq 0.1 K_s$	dependent on relative density and C_u
Carrol (1983)	$O_{95}/d_{85} \leq 2-3$ $K_f \geq (1-10) K_s$	woven and nonwovens
Christopher and Holtz (1985)	$O_{95}/d_{85} \leq 1-2$ $O_{95}/d_{15} \geq 3$ $K_f \geq (1-10) K_s$	Dependent on C_u , from grouting experience

Table 2.4. Recommended geotextile design criteria
(after Fischer et al, 1990)

1. Cohesionless Soils		
(1)	$O_{50}/d_{85} \leq 0.8$	
(2)	$0.8 \leq O_{15}/d_{15}$	$C_u < 4$
	$1.2 \leq O_{15}/d_{15}$	$C_u \geq 4$
(3)	$O_{50}/d_{15} \leq \delta$	see figure below
(4)	$\beta_1 \leq O_{50}/d_{50} \leq \beta_2$	see the figure below
2. Cohesive Soils		
sandy silts and clays	$O_{50}/d_{85} \leq 0.8$	
fine clays	$O_{50} \leq 0.1 \text{ mm}$	
fine silts	$O_{50} \leq 0.06 \text{ mm}$	
very fine soils	$O_{50} \leq 0.04 \text{ mm}$

Developments from Laboratory tests

This approach combines the previous developments of standards for granular filters with extensive testing of geotextiles to improve and modify those standards for geotextile filters. The first criteria of such kind was developed by U.S. Army Corps of Engineers (Calhoun, 1972). Extensive testing of this kind took place in other parts of the world to provide similar criteria, e.g., in England by McGown and his co-workers, in the Netherlands at the Delft Hydraulics Laboratories,

The Corps of Engineers criteria utilizes the AOS of geotextiles and adopts the following criterion for soils with 50% or less passing the No. 200 sieve.

$$AOS < d_{85s} \quad (2.13)$$

For all other soils $AOS < \text{U.S No. 70 sieve}$. The Delft Specifications (Ogink, 1975) consider the following guidelines for soils in contact with sand under steady state conditions, when a self induced filter can be formed.

$$\text{woven} \quad O_{90} < d_{90s} \quad (2.14)$$

$$\text{non-woven} \quad O_{90} < 1.8d_{90} \quad (2.15)$$

Rollin et al, (1985) tested a variety of woven and non-woven fabrics to study soil retention, level of clogging, hydraulic resistance and pore size histogram. The soils were in the silt range with clay content less than 5% and uniformly graded. The

retention criteria was based on the FOS of the geotextiles and the d_{85} of the soils. The criteria is:

$$FOS < 2.5d_{85} \quad (2.16)$$

Faure et al, (1986)^a performed similar long term tests using loose soils mostly in the silt range and obtained the following criteria.

$$\text{critical} \quad FOS < 1-1.2 d_{85} \quad (2.17)$$

$$\text{less critical} \quad FOS < 1.5-2d_{85} \quad (2.18)$$

Qureshi, (1990) presented a similar criteria based on tests on silty soils and suggested it for gap graded soils and for both woven and non-woven geotextiles.

$$FOS < 3d_{85} \quad (2.19)$$

Filter Criteria for Hydrodynamic Flow

In some engineering applications, e.g., offshore, railway and highway applications, the geotextiles are subject to hydrodynamic and reversing flow conditions. Under such conditions, the formation of an internal stable soil filter is doubtful (Hoare, 1982). The piping and clogging phenomena may be severe (Rollin and Lombard, 1988). More conservative criteria are often assigned due to the anticipated uncertainties in the field. According to Ogink (1975), geotextile filters for hydrodynamic flow conditions should be

selected according to the criteria which require the retention of the smallest particles. He suggested the following criterion.

$$O_{95} < d_{15_f} \quad (2.20)$$

The criterion seems to restrictive and difficult (if not impossible) to apply in practice. Ingold, (1985) on the basis of experimental tests, found that for alternating hydraulic gradient less than 1.0, self filtering might occur in soils depending on the coefficient of uniformity of the soil. The following criterion was suggested by Ingold for reversing flow conditions

$$O_{90} \leq d_{50_f} \quad (2.21)$$

Criteria for Highway Subdrainage

Janssen, (1982), reported the use of a dynamic filtration device for simulating field conditions for filter fabrics used in pavement sub-drains (see Figure 2.17). The test was run at University of Illinois and is known as the University of Illinois dynamic filtration test. The equipment used facilitates the application of dynamic loads while passing water through the sample. The soil used consisted of 90% concrete sand and 10% silt. Load repetitions up to one million cycles were applied. Figure 2.18 plots the sample permeability versus accumulated loads. A fluctuation of permeability can be clearly observed. The permeability decreased up to about 650,000 cycles and increased afterwards. This was attributed to the formation (k decreased) and

Figure 2.17. Schematic of dynamic triaxial device used in University of Illinois (after Janssen, 1982)

collapse (k increased) of a graded soil filter. Gradation analysis showed movement of (-#80 to +#200 sieves) sizes close to the fabric and passage of most of the fines (-#200). No conclusions were drawn by the author as to the selection of fabrics using his test procedure.

Koerner and Koerner, (1992) reported the use of another type of a dynamic filtration device at the Geosynthetic Research Institute (GRI), Figure 2.19. The test was intended to simulate actual highway flow conditions where faulted pavements exist. the test equipment differs from the one used at the University of Illinois in that it was designed to generate pulsating flow across the geotextile. The soil was tested in slurry form. The test was described by the authors is quite involved and requires expensive apparatus and skilled technicians. Typical results from the test are shown in Figure 2.20.

Two methods used to simulate geotextile filters used in highway subdrainage have been reviewed. The two approaches, although designed to simulate the same field conditions, differ in the type of equipment, loading, hydraulic conditions, boundary conditions and the type of soil used. This is basically because of the lack of knowledge of the effect of traffic loading on the trapped water inside the pavement and the difficulty in simulating the exact field conditions.

General

Rollin and Lombard, (1988) summarized the five most used filter criteria in the U.S. and around the world for geotextiles.

Figure 2.18. Variation of system permeability with load repetition (after Janssen, 1982)

Figure 2.19 Schematic of GRI dynamic filtration device (after Koerner and Koerner, 1992)

Figure 2.20. Results of the dynamic filtration test at GRI (after Koerner and Koerner, 1992)

The criteria were based on both laboratory testing and experience. The criteria are provided by; the French Committee on Geotextiles and Geomembranes (CFG, see Table 2.5; Franzius Institute of Hanover (FIH), see Table 2.6; Giroud, 1982, see Table 2.7; FHWA, see Table 2.8; and IRGIM/EPM, developed by IRIGM Institute in Grenoble in association with Ecole Polytechnique of Montreal in Canada, see Table 2.9).

From the above review, the following observations indicate some shortcomings in the existing filter criteria.

- 1) The geotextile is described by a single parameter, i.e., AOS, FOS, O_{90} or O_{50} . The overall pore size distribution is not considered nor is the influence of the smaller pores.
- 2) The soil is also described by a single grain size, e.g., d_{85} or d_{50} . Very few criteria consider other soil characteristics such as the uniformity coefficient (Schober and Teindel, 1979) and density index (Giroud, 1982). However, no criterion considers the internal stability or suffusion characteristics of the soil.
- 3) Long term criteria are not well addressed and this fact will be emphasized in the following paragraphs.

Long Term Performance of Geotextiles

Geotextiles must function as filters for many years or throughout the lifetime of a structure. This is possible if no alteration or change in the geotextile structure takes place. The alteration and change can be due to: 1) Accumulation of fines or

Table 2.5. Coefficients for (CFG) Criteria¹
(from Rollin and Lombard, 1988)

Coeff.	Value	Condition
C ₁	1.0	for continuous and well graded soils
	0.8	for continuous and uniform soils
C ₂	1.25	for confined and dense soils
	0.8	for unconfined and loose soils
C ₃	1.0	for hydraulic gradient, $i < 5$
	0.8	for hydraulic gradient, $5 < i < 20$
	0.6	for hydraulic gradient, $20 < i < 40$
C ₄	1.0	for filtration function only
	0.3	for filtration and drainage function

¹ CFG Criteria is $FOS < C_1 \times C_2 \times C_3 \times C_4 \times d_{85}$

Table 2.6. FIH filter criteria

(from Rollin and Lombard, 1988)

Table 2.7. Giroud's filter criteria (from Giroud, 1982)

	$I < C_u < 3$	$C_u > 3$
$I_0 < 35\%$	$AOS < 1.0 C_u d_{50}$	$AOS < 9.0 d_{50}/C_u$
$35\% < I_0 < 65\%$	$AOS < 1.5 C_u d_{50}$	$AOS < 13.5 d_{50}/C_u$
$I_0 > 65\%$	$AOS < 2.0 C_u d_{50}$	$AOS < 18.0 d_{50}/C_u$

Table 2.8. FHWA filter criteria
(after Christopher and Holtz, 1985)

Filtration			
for soils with 50% or less passing #200 (sands)	steady state AOS = $B d_{85}$	$B = 1$ $B = 0.5 Cu$	$Cu < 2$ or $Cu > 8$ $2 < cu < 4$ $4 < Cu < 8$
	dynamic pulsating cyclic flow AOS < d_{15}	$O_{50} < 0.5 d_{85}$	
for soils with greater than 50% particles passing #200	steady state AOS < $0.3 d_{85}$	woven	$O_{95} < d_{85}$
		nonwoven	$O_{95} < 1.8 d_{85}$
	dynamic pulsating cyclic flow AOS < d_{15}		$O_{50} < 0.5 d_{85}$
Clogging			
severe application		gradient ratio > 3 $K_f > 10 K_s$	
non-severe applications	woven	% open area > 4	
	nonwoven	porosity > 30%	

Table 2.9. IRIGM/EPM filter criteria
(after Rollin and Lombard, 1988)

uniform soils	$FOS < 1.5 d_{85}$
well graded soils	$1.5 < FOS/d_{85} < 3.0$

any other matter between or on the fibers, resulting in structural modifications; and 2) Degradation of fibers or loss of bond strength. These modifications can be caused by phenomena such as clogging, blocking, blinding and piping (Rollin and Lombard, 1988).

Clogging, essentially occurring within the fabric structure, can be caused by soil particles, sediments or organic matter migrating between the fibers and becoming trapped. Other causes might be salt deposition, mineral precipitation and bacterial growth. A clogging level may be explained in terms of permeability of the installed geotextile (Giroud, 1982). However, clogging is mostly partial, rather than total, in most of the drainage applications (Rollin and Lombard, 1988).

Another important long term effect is degradation of the fibers and loss of bond strength. The degradation may result from mechanical, chemical, environmental, and bacterial actions. However, chemical degradation is critical when fabrics are used in chemically active environments such as landfills.

In designing geotextiles as filters in drainage systems the long term performance of the drainage system needs to be known and the structural modifications that can take place with time need to be predicted. The existing approaches to the prediction of long term performance involve performing short term tests (long term in the laboratory) and extrapolating the results.

Long Term Properties

Prediction of long term filtration performance is a major concern to designers and users of geotextile filters due to the

complex interaction between the fabric and the adjacent soil. The process is governed by the structural properties of the geotextile, the soil properties and the hydraulic conditions. There are three major approaches that are currently used to predict long term performance, mainly:-

- The gradient ratio test
- Long term filtration test
- Morphological analysis

The Gradient Ratio Test

This test was initiated by Calhoun, (1972) of the Corps of Engineers to evaluate the filtering ability of a geotextile. The apparatus used is similar to the one shown in Figure 2.21. Hydraulic heads are measured at various heights above the fabric in a soil placed in a vertical cylinder. The ratio of the head loss, after 24 hours of the flow, between points close to the fabric and far away from it is defined as the gradient ratio. The test conditions are severe, i.e, a gap graded soil and high hydraulic gradients are used. The U.S. Army Corps of Engineers established a maximum acceptable clogging ratio of 3.0 based on the original tests carried out by Calhoun, (1972) and subsequently verified by clogging tests carried out by Carrol, (1983), and Haliburton and Wood, (1982).

Research by Rosen and Marks (1975), Haliburton and Wood (1982), and Carrol, (1987) revealed that the gradient ratio is a function of the soil type and of hydraulic conditions. Carrol showed that fabric clogging is not significant when the system

Figure 2.21. The U.S. Corps of Engineers gradient ratio device (after Calhoun, 1972)

hydraulic gradient is low (a well graded silty sand was used in his tests with 15% passing No. 200 sieve). He also concluded that neither permeability nor AOS of the fabric tested indicated a relation to the soil fabric system performance. According to Carroll (1987); 1) the test should be set to simulate in-place field conditions; and 2) the soils and hydraulic gradients used should duplicate expected field conditions. Tests which deviate considerably from these conditions will not provide a useful performance evaluation.

Williams and Luettich (1990) cited about 9 limitations for the gradient ratio test. The limitations are:

- 1) no effective confining stress is placed on the soil;
- 2) the soil is rained or tremmied into place, which induce segregation of the soil;
- 3) the initial hydraulic conductivity of the soil in the gradient ratio device may vary by more than 1 to 2 orders of magnitude, depending on the sample placement conditions;
- 4) the structure, density, void ratio of the soil are not controlled or monitored during the test;
- 5) the compressibility, porosity and pore size distribution of the geotextile are not monitored or controlled during the test;
- 6) due to the absence of any back pressure capabilities incomplete saturation of the soil often results in poor reproducibility of the gradient ratio test results;
- 7) the flow velocities, the state of stress, stress history, type of drainage media and change in soil or geosynthetic

properties due to aging, chemical or biological attack, or construction damage can not be modeled by the test method;

8) the gradient ratio test equipment is practical only for soils with hydraulic conductivities greater than about 0.0004 cm/sec; and

9) sidewall piping often results in fictiously high hydraulic conductivity values; particularly with low permeability ($k < 0.0001$ cm/sec) soils, and therefore often results in poor reproducibility of the gradient ratio test results.

Inspite of the above limitations, the gradient ratio test is widely used and a draft for an ASTM standard has been written (Koerner, 1991). The major advantage of the gradient ratio test is that it provides a quantitative index value (the gradient ratio) which serves as a measure of the susceptibility of a geotextile to clogging under severe test conditions.

Long Term Filtration Tests

Due to the limitations of the gradient ratio test, some researchers (Koerner, 1990) believe that flow through soil/geotextile systems can only be definitely examined by long term filtration tests. Koerner and Ko, (1982) presented the results of long term column tests in a vertical flow device, run until equilibrium flow conditions are achieved. A typical long term flow curve of silty clayey soil and non-woven needle punched fabric is shown in Figure 2.22. Two portions of the curve were defined, an initial steep portion and a final flat portion. Koerner and Ko,

Figure 2.22. Long term flow curves for nonwoven geotextiles and four soil types (after Koerner and Ko, 1982)

(1982) concluded that the final range governs the soil-fabric interaction. A zero or nominal final slope is preferred over a steeper one, as the former indicates equilibrium of the soil-fabric structure. A slope markedly greater than zero indicates a non-equilibrium situation, and may suggest blinding of the fabric. However, Williams and Luettich believe that the long term filtration test is subject to the same limitations as the gradient ratio test, plus the impractical long period of testing (more than 30-40 days) to reach equilibrium.

Mlynarek et al, (1991) presented test results from a test that was developed to measure soil retention and water evacuation through a soil geotextile system using a permeameter Figure 2.23. Using two natural soils, Figure 2.24, and a number of geotextiles, three types (type I, II and III) of filtration behavior were defined Figure 2.25. Type I indicates cake build up and soil stabilization; type II indicates continuous loss of particles; and type III indicates cake build up with loss of few particles and soil stabilization. According to Mlynarek et al, (1991) geotextiles can act as a good filter as long as its filtration behavior curve is of types I or III and no more than 5% of the total weight of particles is entrained by the water under the specific test conditions given by Mlynarek et al, (1991).

Morphological approach

The previous methods for evaluating long term behavior do not provide information on the manner and method by which particles are collected in the geotextile structure. Therefore, it is

Figure 2.23. Schematic diagram of EPM filtration device
(after Mlynarek et al, 1991)

Figure 2.24. Particle size distributions of soils used by Mlynarek et al, 1990

Figure 2.25. Selected filtration curves of Ormstown-geotextile system (after Mlynarek et al, 1990)

questionable whether the simulation studies are realistic and suitable geotextiles.

Sotton et al, (1982) provided results of an investigation of the status of 30 geotextiles taken from sites in France that had been in service for more than 12 years. The objective of the study was to understand the mechanisms necessary for partial and complete clogging. Electronic microscopy was used to examine the samples. The samples were dried, impregnated by resin and thin sections were made from the impregnated specimens. Morphological examination of the thin sections revealed that mineral clogging can result from: 1) penetration of suspension of soil into the geotextile; 2) sedimentation of clay particles around the fibers of the geotextile; and 3) the presence of certain chemicals within the soil which helped to promote clogging. Such chemicals are gypsum, iron, sodium and magnesium.

The study showed that even in most clogged geotextiles, there are still open pores and clogging is not complete.

Qureshi, (1990) studied micrographs of non-woven fabrics, tested with gap graded soils in the laboratory. Needle punched and heat treated fabrics were used in the study. The micrograph were made for polished impregnated sections. The analysis of the micrographs showed that the fabrics had not clogged entirely. The degree of clogging was higher for heat treated fabrics.

CHAPTER 3: DESIGN REQUIREMENTS OF PREFABRICATED EDGE DRAINS FOR HIGHWAYS

Introduction

Prefabricated highway edge drains (PFHEDs) are gaining wide acceptance as a viable alternative for conventional highway edge drains. However, because they are new products their reliability has not been well assessed. An efficient PFHED should satisfy two major requirements for its core: 1) it should have adequate flow capacity; and 2) it should have adequate strength to resist the installation and in-service stresses. This chapter addresses the core design requirements. Guidelines have been given for the proper selection of PFHEDs for highways in Indiana.

. Flow Requirements of PFHEDs

In order to evaluate the flow requirements of a PFHED, the amount of water entering the drain "the demand" and the flow "capacity" of the drain need to be known. The designed PFHED should be sufficient to carry all the water that could be released by the base course, i.e., it should have adequate reception and flow capacity.

Estimation of the Demand

Two approaches can be followed to estimate the demand. In the first approach the demand is estimated using empirical equations. In the second, the demand is estimated from the measured flow of successful conventional drains, i.e., it is assumed to be equal to or greater than their actual flow capacity.

Empirical Equations

Two empirical equations will be used to estimate the amount of water entering the edge drain. The first is based on the precipitation rate (Cedergren, 1972), while the second is based on the pavement condition (Moulton, 1980).

Cedergren

This method is based on the design precipitation rate for the locality where the drains are to be installed, i.e., the 1 hour per year precipitation. The amount of water entering the drain q is estimated by:

$$q = c i_r W f_r \quad (3.1)$$

Where:

c is the pavement infiltration coefficient and is 0.5 - 0.7 for concrete pavements

i_r is the rainfall intensity and is 1.2 in/hr in Indiana (Cedergren, 1972)

W is the width of the base course or drainage layer (L)
 f_r is a release factor and is 0.25 for well graded base courses in service for more than 5 years (Koerner, 1989).

Consider a two lane highway ($W = 24$ ft) with a dense well graded base and outlets spaced 500 feet. Assuming the flow to be collected from one side only, the minimum and maximum estimated demand, using Equation (3.1), are 18.7 and 25 gallons per minute for infiltration coefficients 0.5 and 0.67 respectively.

Moulton

This method was developed because it was found that the duration of rainfall is more important than its intensity and infiltration is more related to cracking. The infiltration is estimated by:

$$Q = I_c \left(\frac{N}{W} + \frac{W_c}{WC_s} \right) + K_p \quad (3.2)$$

Where:

I_c is crack infiltration rate, taken as 2.4 ft³/day/ft of the drainage layer

N_c is the number of contributing longitudinal cracks

W_c is the length of contributing transverse cracks or joints (ft)

W is the width of granular base or subbase subjected to infiltration (ft).

C_s is the spacing of transverse cracks or joints (ft).

K_p is the rate of infiltration through un-cracked pavement and is usually neglected.

For a two lane pavement (2-12 feet) with dense asphalt concrete shoulders and 18 feet joint spacing, the infiltration is equal to 0.433 cfd/ft^2 . If a release factor of 0.25 is assumed for the subbase, for an outlet spacing of 500 feet, the demand is 6.73 gallons per minute.

Demand from Measured Flow

In order to accept prefabricated edge drains as viable alternatives to conventional edge drains, they should perform at least as well as conventional drains. Therefore, the idea is to impose a demand on the new drains equal to or greater than the measured field capacity of successful standard circular pipe and granular envelope systems.

Dempsey, (1989) presented results of outflow volumes observed from subdrainage systems in Illinois. The measurements were conducted using a tipping bucket flow meter, at outlets spaced at 500 feet intervals at a test site on FA180 and FA157 near Champaign, Illinois during 1983 and 1984. Flow measurements were made at outlets of both conventional and prefabricated drains. Quantities of over 2.5 gallons per minute were measured from outlets of standard pipe system with a sand envelope. The prefabricated drain system (Hydraway 2000) had flow capacities in

excess of 300 gallons per hour (5 gallons/minute).

Measurements of flow volumes from both conventional and geocomposite edge drainage systems were performed in Indiana for the fall of 1991 (Ahmed, 1992). The measurements were performed on sites in which conventional and prefabricated drains were installed. A maximum flow of 4.88 gallons per minute was measured for a conventional drain on US 31 in Hamilton County. The pavement was a concrete pavement, with no overlay, underlain by a bituminous stabilized base. However for prefabricated edge drains a maximum (5 minute) flow of only 2.83 gallons per minute was measured at a site on US 41 in Sullivan County. The pavement at the latter site was concrete pavement underlain by a bituminous stabilized base.

It is noted that the calculated demand from Moulton's empirical equation gave values very close to the field measurements. However, Cedergren's equation overestimates the flow. Due to the uncertainties in the assumptions made to develop the equations, the field measurements will be taken as a basis for estimating the demand. Therefore, based on field measurements, the demand (the amount of water entering the PFHED) should be taken to be greater than or equal to 5.0 gallons per minute for a prefabricated edge drain to perform better than conventional edge drains. This amount was obtained from field measurements, therefore, the question is how to determine, using an appropriate laboratory technique, a threshold value equivalent to this field value?

The Flow Capacity of PFHEDs

Currently the flow capacity of geocomposite drains is measured using ASTM D4716-87 for "Determining the Constant Head In-plane Flow of Geotextiles and Geotextile Related Products". The method does not simulate the conditions in the field because the tested material is laid horizontally for testing. Dempsey, (1989) used a 7.3 meter long channel in which the geocomposite fin drain is positioned vertically, to estimate its maximum flow capacity (refer to Figure 2.4 in the previous chapter). The test conditions are very close to the field conditions, therefore, it is expected to give a good estimate of the field flow capacity of the drain. However, the major shortcomings of the test is the high cost of building the equipment and the large space needed for it. Therefore a simple test is needed to estimate the actual flow capacity of a PFHED.

The Required Flow Capacity

Dempsey measured the flow capacity for 5 geocomposite fin drains using both the ASTM D4716 and the long channel. A good correlation was found between the two (see Figure 3.1). It is important to observe that the channel-flow values in Figure 3.1 are for 6.3 inches channel entrance head, while the in-plane tests are for an applied pressure of 3.4 psi.

Following the assumption that Dempsey's long channel closely simulates field conditions, the required in-plane capacity which corresponds to a demand of 5.0 gallon/minute (300 gallons/hr) is

Figure 3.1. The relationship between the in-plane and the long channel flow (after Dempsey, 1989)

about 15 and 18 gallons/minute/foot for drain slopes of 2% and 1% respectively (see Figure 3.1). Therefore, depending on the gradient in the field, a drain having in-plane capacity of 15-18 gallons/minute/foot (surcharge of 3.4 psi), is expected to efficiently drain pavements when the subbase is a dense graded material, and will perform better than standard subdrainage system with a sand envelope.

There are three observations that need to be considered:

- 1) ASTM requires a stress of 10 psi be applied during the in-plane test, whereas Dempsey's in-plane tests were run under 3.4 psi. As for this first observation, an acceptance of a flow value for tests performed under 10 psi (ASTM-4716-87) is on the conservative side.
- 2) Dempsey's long channel test results, reported in Figure 3.1, were performed with a entrance head of 6.3 inch. This implies that, if the above estimates for capacity are applied, drains in the field should have a flow depth of at least 6 inches below the subgrade level.
- 3) Outlet spacings for the above estimates and recommendations are a maximum of 500 feet.

Koerner and Hwu (1989) tested 6 widely used PFHEDs for their flow capacity under different gradients and applied pressures. The flow measurements under 5 and 10 psi are reported in Table 3.1. The recommended values are compared with the flow capacity of typical PFHEDs and it can be shown that only the Akwadrain type B failed to satisfy the suggested value of 15-18 gallons per minute. It should

be noted that minimum slopes of 2% and 1% are needed when the drain has in-flow capacities of 15 and 18 gallons/minute respectively. However, the performance of all the drains in Table 3.1 is questionable if they are draining permeable aggregate bases. An important requirement is that at least 6 inches of the drain should be below the subbase and subgrade contact level. This implies that, if the base course is more than 5 inches thick, 12 inch drains should not be used for 500 ft spacing.

From the above, it is concluded that in order for a prefabricated edge drain to perform equally or better than a conventional edge drain when draining dense subbases, it should have a minimum inplane capacity of 15 to 18 gallons per minute (by ASTM D4716-87), depending on its longitudinal slope in the field.

Strength Requirements of PFHEDs

Introduction

Structural-related problems of PFHEDs have been reported from different states, e.g., Kentucky (Fleckentein and Allen, 1991), New York and Michigan (Frobel, 1991), and from West Virginia (Baldwin, 1991). Problems observed are damaged core cuspatations, J'ing of the core, compression of the core and damage to the flexible outlet pipes. These problems have been attributed to improper construction practice, overstressing during compaction, eccentric in-service loading and in-service compression. The structural defects are critical to the performance of the PFHED because they tend to decrease the rate of flow and the flow capacity of the drain.

Table 3.1: Typical flow capacities of PFHEDs
(after Koerner and Hwu, 1989)

Drain type	Flow Capacity (gallons/minute)	
	10 psi ¹	5 psi
Hydraway 2000	16	16.5
Contech Stripdrain 100	15	16
Akwadrain Type B	9	10
ICE Akwadrain 25	15	16
ADS	17	17.5
Nilex Nudrain	15	16

¹ASTM D4716-87

In order to assess the structural requirements of a PFHED it is necessary first to estimate the types and amount of stresses to which a PFHED system is subjected, and then to compare these stresses with the strength of the PFHED component as obtained using appropriate testing methods. The strength should then exceed the applied stresses by an appropriate margin of safety.

In this section an overview of the construction practices for PFHEDs are given, with special emphasis on the practice in Indiana. The section also contains quantitative evaluation of the stresses to which PFHED components are subjected during construction and throughout the lifetime of the drain. This is followed by a guideline specification for the strength requirements of PFHEDs.

Edge Drain Construction

A PFHED is usually located adjacent to the pavement under the shoulder so that water entering the pavement/shoulder joint can drain rapidly. In cases where a tied PCC shoulder exists, the edge drain should generally be located along the outside edge of the shoulder. In Indiana, the edge drain is installed according to the manufacturers specifications or as directed by the engineer. The following steps are followed:

- 1) A trench 4-6 inches wide is excavated in the appropriate location. The speed of the trenching operation is controlled at a rate low enough to ensure that the mixture of the excavated material is fine enough to later facilitate proper compaction during trench backfilling.
- 2) The trench backfill is placed in layers 8 inches maximum

loose measurement and the first 8 inch is placed simultaneously with the installation of the edge drain (Figure 3.2).

3) Each layer of backfill is compacted by one pass of a vibratory plate compactor operated at 500 cycles per minute and at a maximum speed of 50 feet/minute. The compactor is required to have a minimum impact force of 5000 pounds and shall be equipped with a modified compactor plate according to the edge drain manufacturer's specifications (the one specified by Monsanto has a plate area of about 50 square inches). A rolling wheel that fits the space between the geocomposite and the trench edge is sometimes used for compaction. All portions of the excavated materials should be backfilled and compacted.

4) The bituminous cap is then placed and compacted according to the appropriate requirements.

Induced Stresses

Construction-Related Stresses

Assuming proper and careful handling of the drain during the different stages of construction, the most critical construction-related stresses are those induced during compaction. Compaction induced stresses have been held responsible for damages to edge drains (Fleckentein and Allen, 1991).

A clear explanation of the behavior and characteristics of a clayey soil due to compaction can be explained by the deformable

Figure 3.2. A typical installation process of a PFHED

aggregate model (Hodek, 1972). The model suggests that, prior to compaction soil particles are grouped in agglomerations or peds whose size and strength characteristics are influenced by the soil water content. During compaction, at low water content (below optimum), these peds have a high strength and are better able to resist the compaction pressures without much distortion. Thus there exist two networks of pore space in the clay mass, a network of large inter-aggregate pores and a network of small intra-aggregate pores. As the molding water content increases (dry of optimum) the aggregates decrease in strength and suffer greater deformation during compaction. This results in a decrease in inter-aggregate pores and increase in the dry density. The highest degree of compaction can be obtained when the moisture content is at optimum. With increasing moisture content above optimum, aggregates are easily distorted and fused together. Reorientation and dispersion of individual soil particles may take place. The above model shows that the energy needed to induce proper compaction becomes a function of the type of soil and its compaction moisture content.

Converse, (1956) showed that for a moderately cohesive soil it is necessary to provide a compressive force at contact of at least 35-100 psi in order to induce plasticity, and that greater forces are needed for effective compaction. However, for best results the compactor should provide a dead load on the order of 10-12 psi. With oscillators and vibratory compactors, conditions are favorable for dynamic amplification of contact pressures.

For the installation conditions according to the specifications in Indiana, it is necessary to have a vibrator that

can generate an impact force of 5000 pounds. If this force is exerted on a plate area of 50 in², then the contact pressure is 100 psi. This value agrees with the estimated compressive stresses (Converse, 1956) needed for proper compaction of moderately cohesive soils.

The horizontal component of the estimated compaction stresses transferred to the drain, σ_h , is given by:

$$\sigma_h = K_o 100 \quad \text{psi} \quad (3.3)$$

K_o ranges between 0.4 and 0.5 for normally consolidated soils (Holtz and Kovacs, 1981). If we assume the soil to be slightly prestressed after compaction, an average K_o value of 0.45 is considered acceptable. The horizontal component of the compaction induced stresses at contact is estimated to be 45 psi. This value is probably higher than the actual applied pressures because the compressibility of the drain will reduce the K value.

At the soil/geotextile contact, the limiting tangential stress is equal to $\sigma_h \tan(\delta)$, where δ is the soil/geotextile friction angle (Figure 3.3). The value of δ was measured by Williams et al, (1987), and was found to be equal to 37° for nonwoven fabric/glacial till contact. If the flexibility and compressibility of the drain are considered, the actual δ value will be much less than 37°. For conservancy, a value of 37° is adopted. Therefore;

$$\sigma_v = \sigma_h \tan 37 \quad (3.4)$$

The resultant stress σ_r will be:

Figure 3.3. Stress conditions on a PFHED

$$\sigma_r = \sqrt{33.9^2 + 45^2} = 56.3 \quad = 8114 \text{ psf} \quad (3.5)$$

The In-Service Stresses

The most critical condition after backfilling is when a sustained load (e.g., a truck), parks above the edge drain (Figure 3.4). Boussinesq elastic theory can be used to approximate the maximum stress values. Koerner and Hwu (1990), calculated the maximum horizontal stress to which an edge drain is subjected when a standard AASHTO wheel load of 8 tires at 5000 lb each is applied. A maximum horizontal load of 3020 psf or (21 psi) was obtained at a depth of 7 inches. Vertical stress, σ_v , is given by:

$$\sigma_v = 21 \tan 37 = 15.8 \text{ psi} \quad (3.6)$$

$$\sigma_r = \sqrt{15.8^2 + 21^2} = 26.32 \quad = 3787 \text{ psf} \quad (3.7).$$

The stress σ_r is applied at an angle (δ) of 37° to the horizontal. The value of δ is expected to be much less than 37° due to the flexibility and compressibility of the drain.

The Puncture Stresses on the Geotextile

To support the backfill soil from one core protrusion to the other, the geotextile has to resist stresses caused by the geotextile being pushed into the drain void. This action produces

Figure 3.4. Elevation and plan view of a truck parked right over the drain (after Koerner and Hwu, 1989)

tensile and puncture stresses in the geotextile, around the cuspatations. The puncture mode is more critical than the tension mode (Koerner and Hwu, 1990). The required puncture stress is simply given by the following equation.

$$P_d = \sigma_r (A_t - A_s) \quad (3.8).$$

Where

P_d is the required puncture stress

σ_r is the maximum stress imposed on the geotextile/core system by the backfill

A_t is the geotextile area between centers of adjacent core protrusions.

A_s is the area of the protrusion head

The value of P_d therefore depends on A_t and A_s values, assuming σ_r to be fixed. For typical prefabricated drains (e.g., Hydraway 2000) the spacing between cuspatations is 1.3 inches and the protrusion diameter is 0.4 inch, which render A_t and A_s values of 1.69 and 0.126 in², respectively. The value of P_d , for Hydraway 2000, will be:

$$P_d = 1.564\sigma_r = 1.564 \times 26.3 = 41.1 \quad \text{psi} \quad (3.9)$$

Design Specifications

Specifications for the Compressive Strength of the Core

It appears from a review of the literature on compressive strength test that there are two types of tests used to evaluate the compressive strength of the core, a direct compression test and an inclined load test (see Chapter 2). The response of some drain types, however, varies dramatically with the test method. The questions that come to mind are; is it necessary to consider load inclination? And if so, what angle of inclination is close to field conditions?. The answer to the first question is yes, load inclination should be considered due to the existence of two components of load and due to the roughness of the wrapped fabric. However, due to the compressibility, flexibility, and the small height of the drain, the assumed friction angle δ of 37° is certainly high. A load inclination of 50% or less than this value may be appropriate, say 10 degrees. The inclination factor should be considered only for sustained loads and not for dynamic compaction loads. Based on the above calculations and the subsequent observations and discussions, specifications will be suggested for the compressive strength of the geotextile based on compaction and sustained load resistance.

Based on Compaction stresses

The compaction induced stresses were calculated to be equal to 8114 psf. Due to the short duration of loading during compaction, no factor of safety is to be applied to this value and it is

considered as a required strength for the drain in order to resist the compaction induced stresses. Any appropriate normal compressive test may be used to satisfy this requirement.

Based on Sustained Loads

For sustained loads a factor of safety of 2 is applied in order to obtain a required design strength. This factor of safety accommodates the effects of overweight vehicles, impact loads, long term creep loads and other durability-related factors. A required strength of 7574 psf (i.e., twice the value of σ_c ; Equation 3.7) is needed for the PFHED to resist sustained loads. This value should be obtained at an inclined angle of 80° .

In conclusion, for a PFHED to resist the induced compressive stresses, it should have a minimum normal compressive strength of 8100 psf (at 90°) and an inclined compressive strength of about 7500 psf (at 80°). The direct compressive strength may be obtained using the GRI method (See Chapter 2).

Specifications for the Puncture Strength of Geotextiles

The calculated puncture stress for a typical PFHED fabric is 41.4 psi (Hydraway 2000). If a factor of safety of 1.5 is adopted to take care of variability in the material properties of the geotextile and non-compatibility of the test method, a design puncture resistance of about 62 lb is required for this drain. This value may be obtained using ASTM D4833-88 due to the close similarity between the stress states in the field and when using

this test method. For other drains, the required puncture stress can be calculated using Equation (3.8). A factor of safety of 1.5 is applied to the calculated P_d in order to obtain design puncture resistance.

Specifications for Other Geotextile Properties

Guidelines for some other geotextiles properties that are related to strength can be extracted from "Specification Guidelines for Drainage Geotextiles" approved by Task Force #25 (AASHTO-ARBTA). These include specifications and test methods for grab tensile strength, burst strength, trapezoidal tear strength, seam strength and specification for ultraviolet degradation (Table 3.2).

Field Evaluation of PFHEDs in Indiana

Since 1984 prefabricated edge drains have been regularly used in Indiana as part of the highway rehabilitation program. Hydraway 2000 is the only PFHED widely used in Indiana. However, up to now, no serious investigation has been undertaken to evaluate the performance of PFHEDs in Indiana, and there is no state developed specification for their selection.

The only inspection made for PFHEDs in Indiana was carried out by Monsanto in 1990. It involved borescoping at 27 sites (Kearns, 1992) to evaluate the in place conditions of the drains. No problems were reported. However, due to the lack of information as to the conditions and functioning of the drains, it was decided to

Table 3.2: Physical requirements for drainage geotextiles
(after Task force #25, AASHTO-ARBTA)¹

property	drainage ²		test method
	Class A ³	Class B ⁴	
grab strength lls	180	80	TF 25 #1
elongation %	N/A	N/A	TF 25 #1
seam strength lbs	160	70	TF 25 #1
burst strength lb/in ²	290	130	TF 25 #3
trapezoidal tear lbs	50	25	ASTM D4553
ultraviolet degradation at 150 hrs	70% strength retained	70% strength retained	ASTM D4355

¹from Koerner (1990)

²Minimum - Use value in weaker principal direction. All numerical values represent minimum average roll value (i.e., test results from any sampled roll in a lot shall meet or exceed the minimum values in the Table) - Stated values are for non-critical, non-severe applications. Lots sampled according to ASTM D4354

³Class A drainage applications for fabrics are where installation stressess are more severe than Class B applications, i.e, very coarse sharp angular aggregate is used, a heavy degree of compaction (95 % AASHTO T99) is specified or depth of trench is greater than 10 ft.

⁴Class B drainage applications are those where fabric is used with smooth graded surfaces having no sharp angular projections, no sharp angular aggregates, compaction requirements are light (< 95% AASHTO T99), and trenches are less than 10 feet in depth.

undertake a limited field investigation as part of this research. This included investigating the overall condition of the drain, the condition of the core and checking the wrapped geotextile for possible clogging and caking. The information drawn will help define existing problems and improve construction techniques.

The Investigation Program

The investigation program included the following steps:

- 1) Information was gathered on the locations where PFHEDs were installed in Indiana.
- 2) Three sites were selected on I-65 for inspection: site 1 - has drains installed in 1984; site 2 - has drains installed in 1986; and site 3 - has drains installed in 1988. Drains were exhumed at two locations for each site, first at a location where the pavement was in a good condition and then at a location where there was an apparent transverse crack.
- 3) A hole about 3x3 feet in area was excavated adjacent to the pavement shoulder down to the subgrade. Excavations were made using a backhoe and also manually. When the excavation was completed, the drain was visually inspected and a sample was removed and taken to the laboratory for further investigations.

Sites Investigated

The field investigation was carried out in the fall of 1991. Site 1 is located on I-65 South of Indianapolis between the

Greenwood and Whiteland interchanges. These are the first drains to be installed in Indiana on an experimental basis. Site 2 is located on I-65 in Boone County north of Indianapolis, while Site 3 is located on I-65, south of Indianapolis, between the intersections with US 31 and SR 252. Two locations were excavated at the first two sites, at a location where there is no crack and another where there is crack, while excavations were made at only one location at site 2.

At all of these sites excavations were first made using a backhoe to remove the asphalt layer and the soil around the drain. The soil close to the drain was removed manually. After the soil was removed, the drain was inspected and a sample of the drain was cut for further studies. Photographs were taken and samples from the subgrade and the soil around the drain were collected. The removed portion of the drain was replaced with a new piece which was taped to the old drain.

Results

Only the investigations conducted on the drain will be presented in this chapter. Investigations on the fabric will be presented in Chapter 6.

Site 1 - 1984

The PFHED at this site was 18 inches wide. It was installed about 6 inches into the subgrade. The drain at the first location (no crack) was found to be in good condition, well aligned with no

structural damage, Figure 3.5. The drain was wet but no water was found "standing" in the drain. The second location (adjacent to the crack) was about 100 feet from the first one. After removal of the surrounding soil, structural damages were observed. The drain was compressed, Figure 3.6. When cut, a layer of fine material about 3 inches thick (more than 80% passing #200 sieve) was found inside the drain, Figure 3.7. The subgrade soil is classified as CL.

An effort was made to locate the outlet pipes at this site. The search was made for hundreds of yards on both sides of the excavations. No outlets were found. The outlets were probably buried in the soil or covered by grass. A good practice is to put signals (posts) at outlet points and to routinely clean the outlets in order to allow outflow.

Site 2 - 1986

The drain at this site was 12 inches wide and was installed in 1986. Excavations revealed that the drain was well aligned and has no structural defects. The drain was wet, the outlets were clean and the soil around them was wet, which was an indication of a functioning system.

Site 3 - 1988

The PFHED at this site was 12 inch wide and was installed in 1988. It penetrated less than 3 inches into the subgrade. The drain was found to be in a good condition at the first location (no crack) except for minor J'ing, Figure 3.8. At the second location

Figure 3.5. A well aligned drain at site (1)

Figure 3.6. A compressed drain at site (2)

Figure 3.7. Fine materials piped into the drainage core.

Figure 3.8. Minor structural defect, J'ing, site (3)

(adjacent to a crack) the excavation revealed severe structural damage. The damage was in the form of J'ing and compression of the core, Figure 3.9. The measured drain height was only 9 inches, Figure 3.10. The drain, at both locations, was completely dry and no sign of water was observed.

Summary

From the results of the field investigations the following observations are made:

- 1) Out of 5 locations inspected, structural damages were observed in three locations. Severe structural damage was observed at one location.
- 2) Heavy siltation was observed at one location. This was a clear indication of the inefficiency of the geotextile filter in retaining fine materials.
- 3) Structural damages and siltation were found at locations adjacent to cracks. However, these phenomena can not be related to the existence of cracks due to the limited number of sites excavated.
- 4) The drain at site 2 was completely dry and it is questionable whether the drain at this site was receiving any water.
- 5) The outlets were found covered with grass and/or soil.

Lessons Learned from the Field Investigations

The following measures need to be taken in order to avoid

Figure 3.9. Severe J'ing of the drain, site (3)

Figure 3.10. Severe j'ing caused shortening of the drain, Site (3)

problems that will cause malfunctioning of the drain.

1) Drains should be selected to satisfy certain hydraulic and strength requirements. Guidelines have been given in this chapter for these requirements.

2) Construction control is a necessity. Therefore a borescope evaluation is suggested as part of the specifications for construction. The borescope evaluation will help to detect problems, on site, and suggested measures to improve the construction practice can follow.

3) More attention should be given to the selection of the fabric. A better understanding of the structural properties and filtration characteristics of non-woven geotextiles is needed. A deficient geotextile will either lead to the passage of fines to the drainage core, thus clogging the core space, or to the trapping of fines in the pore spaces of the fabric, therefore inhibiting flow to the core.

4) Outlets should be marked with posts or rails and regularly cleaned, at least once a year.

CHAPTER 4: DETERMINATION OF THE PORE PARAMETERS OF NON-WOVEN GEOTEXTILES

Introduction

Several methods have been introduced to measure pore size parameters (Chapter 2). In the United States the apparent opening size (AOS) is the most widely used pore parameter for the design of geotextile filters. However in Europe the FOS is gaining wider acceptance. Due to the limitations of AOS and the filtration opening size (FOS) (Chapter 2), more refined methods have been recently introduced to measure the overall pore size distribution (PSD) such as mercury intrusion porosimetry and image analysis. Although the pore size concept has been advanced in recent years by the introduction of the above methods, it needs more evaluation and research in order to obtain the maximum benefit from it. Increased use of these methods and more accurate determination of the pore size distribution will pave the way towards a better understanding of both the long and short term performances of geotextile filters.

Pore size distribution was measured for 14 different fabrics using mercury intrusion porosimetry and morphological approaches (image analysis). An advanced image analysis system (Quantiment 570) was used to study the micro-structural properties of the fabrics and measure their pore size distribution (PSD). The objective of this investigation is to develop adequate procedures

by which pore size and structural parameters can be measured using mercury intrusion porosimetry and image analysis, and to assess and compare these procedures with the widely used AOS and FOS methods. This will help in understanding the role of pore size and structural parameters in the filtration behavior of non-woven geotextiles.

The Geotextiles Studied

Fourteen non-woven geotextile were studied. The geotextiles were collected from 4 major manufacturers. At least 3 types of geotextiles of different thicknesses, were obtained from each manufacturer. The geotextiles from Hoechst Selandese (TR-fabrics), Amoco (A-fabrics) and Polyfelt (TS-fabrics) are needle punched, while those from Phillips are needle punched-heat treated. Another very thin (thickness less than 1 mm) non-woven fabric (Tynar 3341) was also selected and tested. A list of the geotextiles and their hydraulic and mechanical properties is given in Table 4.1.

Mercury Intrusion Porosimetry

A general review of the method has been given in Chapter 2. The method is capable of obtaining the complete pore size distribution of a non-woven geotextile and depends on: 1) the experimental measurement of the volume of mercury entering the pore of a submerged sample as the absolute pressure is increased; and 2) the relation between pore size (d) and the pressure required to overcome surface tension forces and to force mercury into the pores of the sample. Satisfactory estimates of the surface tension of

Table 4.1: The engineering properties of the geotextiles

Fabric	Type	Weight (oz/ft ²)	AOS Sieve No	Thickness (mm)	FOS (µm)	Permittivity (Second ⁻¹)	Grab Strength, (lbs)	Grab Elongation (%)	Tear Strength (lbs)	Puncture (lbs)	Ultra- Violet (%)	Polymer Type
Test	-	ASTM	ASTM	ASTM	-	ASTM	ASTM	ASTM	ASTM	ASTM	ASTM	-
Method		D3776	D4751	D1777		D4491	D4632	D4632	D4533	D3787	D4355	
TS600	NW NP	6.0	60-100	2.03	95	2.00	170	>50	75	85	-	polypropylene
TS700	NW NP	8.3	80-120	2.67	83	1.60	225	>50	100	115	-	polypropylene
TS800	NW NP	12.0	100-140	3.30	-	1.20	325	>50	130	150	-	polypropylene
TS1000	NW NP	16.2	>140	4.10	-	0.80	470/350	90/95	170/140	70	-	polypropylene
Ph4	NW NP HT	-	70-100	0.81	105	1.97	120	65	50	65	-	polypropylene
Ph9	NW NP HT	-	70-100	2.30	100	1.48	275	70	110	430	-	polypropylene
Ph12	NW NP HT	-	80-120	2.67	117	0.98	345	75	135	170	-	polypropylene
PR1114	NW NP	4.2	70-100	1.65	195	2.71	135/110	70/85	60/50	60	-	Polyster
TR1125	NW NP	7.4	70-120	2.79	103	2.04	270/225	75/85	105/95	115	-	Polyster
TR1145	NW NP	13.5	100-140	4.44	80	1.22	500/425	90/95	175/170	175	-	Polyster
A4545	NW NP	-	>70	-	-	2.00	90	50	45	65 ¹	70	polypropylene
A4553	NW NP	-	>70	-	-	1.50	200	50	80	130 ¹	70	polypropylene
A4561	NW NP	-	>70	-	-	0.70	325	50	130	250 ¹	70	polypropylene
T3341	NW NP	3.3	60-70	-	-	1.10	120	60	40	30 ¹	70	polypropylene

¹ASTM D4833²Abbreviations used are: NW - Nonwoven; NP - Needle Punched; HT - Heat Treated.

mercury, T , and the contact angle between mercury and the intruded material, θ , are needed (see Equation 2.1), in order to generate acceptable results. Values of T have been well established (Ritter and Drake, 1945; and Prapaharan, 1989), while θ needs to be measured between mercury and the geotextile material. The geotextiles tested were made of either polyester or polypropylene.

Measurement of Contact Angle

The advanced contact angle (θ) between polyester and mercury was measured by (Prapaharan, 1989) and was found to be equal to 135.5° . The contact angle between mercury and polypropylene was measured by this investigator using a contact angle measurement device. A small piece of mercury (about 2 mm in diameter) was dropped on a solid piece of polypropylene. The contact angle was directly measured through the eyepiece of the device. Ten measurements were taken for different drops and an average value of 137.4° was obtained with a standard deviation of 2.4° . An attempt was made to measure the contact angle between mercury and non-woven geotextiles, but failed due to intrusion of mercury into the pores of the geotextiles.

Test Equipment

The AutoPore II, 9220 System was used to measure the volume distribution of pores in the geotextiles. The system, Figure 4.1, is composed of:

- 1) Intrusion, extrusion and detection components. These are

Figure 4.1. AutoPore II, the equipment used for MIP tests

comprised of penetrometers, capacitors and transducers for volume measurements. Wide bore penetrometers (volume = 7.5 ml) were used for most of the geotextile samples because of the large pore volume.

2) Low and high pressure generation and measurement components. For this application only low pressure generation was used due to the relatively large pore diameters of geotextiles.

3) A vacuum system.

4) A mercury system which includes a mercury storage reservoir and mercury de-aerator and sample preparation stations. The sample preparation stations (four for AutoPore II) are used to mount the penetrometers for evacuation, filling with mercury and for low pressure intrusion.

5) Computer system which controls the operation and is used to collect and reduce the data.

Because geotextiles are light weight materials, and to avoid errors resulting from the mercury pushing the fabric towards the penetrometer walls, a sample holder was used to enclose the geotextile specimen (Prapaharan, 1989). The sample holder is composed of two perforated plates made of stainless steel and provided with side holes and screws. The perforations are large enough not to affect the test results.

Test Procedure

A small piece of geotextile (1-2 cm²) was cut and then weighed using a sensitive balance (0.0001g). It was then put in the sample holder which was carefully tightened so that the geotextile would not be compressed. The holder was then placed into the penetrometer which was sealed and placed in one of the sample preparation stations. Two samples were prepared and tested for each geotextile.

The following information needs to be prepared and fed into the computer before a test is started: the evacuation time, evacuation pressure, filling pressure, the maximum intrusion volume and a pressure table. A time of 15 minutes was found satisfactory to evacuate the sample under an evacuation pressure of 15 psi. The selection of an appropriate filling pressure is very important because the filling pressure should neither cause premature penetration nor isolation of the sample. A filling pressure will be considered appropriate if the observed intrusion volume between it and the first pressure step in the pressure table is found to be small. Here, a filling pressure of 0.5 psi was used (which corresponds to a pore size of 390-410 microns). This selection was considered appropriate because insignificant intrusion was measured between this filling pressure and the first pressure step (0.60 psi). A pressure table was prepared before running the test.

The tests were performed using the maximum intrusion volume option. The pressure table option was tried first but difficulties were encountered in creating a table with very small pressure

intervals. Some pressure points were skipped when performing the test by the pressure table option due to the inter-connection of pores. However, the maximum intrusion option allowed the instrument to inject a controlled volume of mercury in each step and to determine the corresponding intrusion pressure. A controlled volume of 0.1 to 0.2 ml/gram, depending on the total pore volume, would allow the generation of more than 30 data points. However a pressure table was still needed in order to select the initial and final intrusion values which were taken as 0.6 psi and 20 psi, respectively. Zero equilibrium time was used. The tests were run under low pressure intrusion only.

Material T3341 is a very thin geotextile (less than 1 mm), therefore, it has a very small pore volume. Three pieces of this fabric were tested at the same time. The pieces were separated by steel meshes with wide openings. To make sure that the meshes would not affect the results, they were intruded with mercury and the filling pressure was found sufficient to intrude their pores.

The test steps included sample evacuation, filling with mercury, stepping pressure generation and data collection. Data collection and reduction were made by the computer using a software provided by AutoPore II.

Pore Size Distribution of Compressed Geotextiles

The ability to determine the pore size distribution of compressed geotextile is a major advantage of the mercury intrusion porosimetry method. The geotextile specimen was compressed to a known deformation between the perforated stainless steel plates,

which were used as a sample holder. The deformation was adjusted with the screws to a specified strain. The remainder of the test procedure was similar to that of un-compressed sample.

Compressibility tests were performed on 3 geotextiles (TS800 Ph12 and A4553) in order to determine their compressibility and stress-strain relationship. The tests were stress controlled and performed using the MTS machine. A small piece of fabric (2.8 inch diameter), was compressed at a rate of 0.33 pound per second for about 9 minutes. Readings were recorded each 10 seconds for the first minute and each 20 seconds for the rest of the test. A seating load of 0.22 pound was applied before running the test.

Presentation of Data

Data collected from the MIP test permitted the determination of the porosity and the specific surface area in addition to the volumetric pore size distribution. Typical data in a reduced format are given in Appendix (A). The cumulative pore size distribution was determined by plotting on an arithmetic scale the pore diameter (d) versus the cumulative percentage of pore volume smaller than (d), given by:

$$\text{cum. \% of pore vol. smaller than } d = \left(1 - \frac{V}{V_t}\right) 100\% \quad (4.1)$$

Where:

V_t is the total intrusion volume in ml/s

V is the cumulative intrusion in ml/s

An arithmetic "normal" scale was chosen because of the narrow range of pores in fabrics. The differential pore size distribution was plotted on a normal scale using the relationship given by Ritter and Drake, (1945).

$$D(r) = 2 \frac{p}{d V_t} \frac{dV}{d p} \quad (4.2)$$

Where:

$D(r)$ is the differential pore size distribution

p is pressure causing intrusion (absolute)

dV is incremental intrusion (ml/gram)

dp is incremental pressure (absolute)

For the compressed fabrics, the differential pore size distribution was not normalized to the total volumetric intrusion, in order to assess the relative reduction of pore volumes for various intruded pore sizes.

The porosity (n) was obtained from PSD data, knowing the specific gravity (G_s) of the geotextile and using the following relationship:

$$n = \frac{V_t}{V_t + \frac{1}{G_s \rho_w}} \quad (4.3)$$

where ρ_w is the density of water. Typical specific gravity values for geotextile fabrics are 0.91 for polypropylene fabrics

(Koerner 1990) and 1.36 for polyester fabrics (Prapaharan, 1989).

The specific surface (SSA) is calculated by AutoPore software by numerically integrating the following equation

$$SSA = -\frac{1}{m \cos(\theta) T} \int_0^{V_t} p \, dv \quad (4.4)$$

where m is the mass of the sample and T is the surface tension of mercury.

Image Analysis - Quantimnet 570 Image Analysis System

An image analyzer system extracts and processes quantitative information from an image. Image analysis consists of a number of stages which reduces the information content of a certain image to pertinent measurements which describe selected components of the image. For this purpose, an optical image is converted by the image analysis system from a macroviewer or a microscope into an electronic signal suitable for processing.

The image analysis system used in this study is a Quantiment 570 manufactured by Leica Cambridge Ltd. The system is provided with an extensive range of image analysis facilities which operate based on mathematical morphology. The facilities include binary, grey and color image processing and measurements. A control software (QUIC) provides rapid interactive operations by means of menus and a mouse. Special functional operations can be performed using QBasic programs. An overview of the system is shown in Figure 4.2. The major parts of the system are, the macroviewer, the

Figure 4.2. The Quantimet 570 image analysis system

computer control display monitor and the image display monitor.

The digital conversion of an image involves dividing the image into pixels or picture points (512 x 512 for an image). A grey level is assigned for each pixel depending on its illumination. Black corresponds to a grey level of 0, while, white corresponds to a grey level of 255. Each pixel is represented by an 8 bit digital word upon which morphological processing may be carried out before detection and subsequent measurements.

The main stages of image analysis consists of 5 steps: 1) producing a grey image; 2) grey image processing, i.e., image transformations; 3) a detection stage which results in producing a binary image; 4) storing the binary image; and 5) performing binary image processing and measurements.

The original image should have a high contrast and minimum or no shading. The image is usually processed using transform functions to remove unnecessary details and noise, and then the objects of interest are detected. Pixels for inclusion in the binary detected image are selected by applying thresholds to the transformed grey image. The detected image (binary) is displayed as a red overlay on the image monitor. The detected image is then edited and amended to remove unwanted features, to include undetected features, and to connect or separate features. Measurements are performed on the final "improved" binary image which represents the objects of interest in the original image.

Measurements are made in a measurement frame which is within the detected image frame. The measurement frame can be adjusted and moved to measure the part of the image of interest (Figure 4.3). To

remove measurement errors that may result from features cut by the measurement frame, Quantiment 570 defines a guard region or border around the measurement frame which is determined by the feature measurement logic algorithm (Figure 4.3). Features are included if their lowest right hand corner lies within the measure frame but not otherwise.

Two kinds of measurements can be performed on the processed binary image, field and feature measurements. The data obtained from field measurements represent the structure of the material as a whole rather than of discrete objects. Feature measurements however produce data related to each isolated object within the measure frame. Quantiment 570 offers a wide selection of measured parameters which may be arithmetically combined and the results calibrated to solve a wide range of applications. Results will be given in pixels unless a calibration is performed for the image under consideration. Among the parameters offered by Quantiment 570 are Feret derived parameters. A Feret is the distance between two parallel lines, in a specific direction (Feret angle), which touch an objects at two opposite sides (Figure 4.4). Up to 64 Ferets can be measured by Quantiment 570, at different angles, for one feature.

Pore Parameters by Image Analyzer

Attempts were made in the past to indirectly determine the pore size distribution of non-woven geotextiles by an image analyzer (Prapaharan et al, 1989; Masounave et al, 1980). Prapaharan used Scanning Electron Microscope (SEM) photographs to

Figure 4.3. Illustration of the image and measure frames of the image analyzer

Horizontal Feret

Vertical Feret

Feret 22.5 and 112.5

Figure 4.4. Ferets shown at different angles

directly measure the PSD. However, Masounave et al used cross sections of impregnated fabrics to measure the fiber density, and hence used a probabilistic model based on fiber density to determine the pore size distribution of non-woven geotextiles thicker than 1.5 mm (refer to Chapter 2 for more details).

In this study SEM photographs were used in an attempt to determine PSD, pore shapes and the effect of manufacturing process on the geotextile structure, while thin sections were used to study the internal geotextile structure, i.e., determine the PSD, porosity and fiber spacing. Attempts to measure the PSD following the methods used by Prapaharan (1989) and Masounave (1980) were not successful, and were accompanied by significant shortcomings, as will be explained later. Therefore, a new approach was used to determine pore size parameters from thin sections (microscopic photographs). The new approach measures a set of one-dimensional parameters which were used to characterize the pore structure of non-woven fabrics.

Pore Parameters from SEM Photographs

A typical SEM photograph for a geotextile surface is shown in Figure 4.5. Surface analysis was made for representative geotextiles, one from each group (TS600, TR1114, Ph9, T3341 and A4545). The objective was to measure the PSD, surface porosity and shape parameters for the detected pores. Enlarged SEM photographs (Figure 4.6 - 270 times) were taken for the above geotextiles in order to measure their fiber diameter, a parameter considered by some investigators as important and affecting the filtration

Figure 4.5. An SEM photographic print of Amoco fabric

Figure 4.6. An enlarged (270 times) SEM print of Phillips fabric

behavior (Faure et al 1986; and Rigo et al 1990).

The measured field parameters of interest were area, fill area and the number of features measured. The feature parameters were breadth, width, equivalent circle diameter, roundness and fullness ratio (parameters are explained in Table 4.2).

The accuracy of the measurements using an image analyzer was found to be a function of the quality of the photograph. Sufficient color contrast was needed between the voids and fabric in order to obtain good results. Difficulties were encountered in getting high contrast SEM photographs due to the effect of coating (the geotextiles were coated with aluminum before being scanned). The SEM photographs had significant three dimensional (depth) effects which made the determination of the PSD a difficult, if not impossible, task (Figure 4.5). The lower layers subdivided the pores and resulted in uncertain results.

Negatives were used for image analysis as they were found to give better digitized images than the photograph prints. However, a series of transformations were performed on the digitized images before detection in order to reduce the depth effect.

Pore Parameters from Thin Sections

The thin section analysis was preceded by chemical impregnation of the geotextile. The following steps were followed for sample preparation.

- 1) A sample about 3 cm by 3 cm was placed in a cup covered with a thin layer of the impregnating material (3M electrical

Table 4.2. Parameters of interest, measurable by Quantiment 570

Measurement	Description
1. Area (A)	1. A is the number of pixels in an object
2. Area Fraction	2. is the ratio of the detected field to measured frame area
Perimeter	Measures the total length of boundary
Feature count	Q570 uses lowest right hand point of the image which lies within the measure frame
Length	Length is the value of the maximum Feret
Breadth	the value of the minimum Feret
Anisotropy	The ratio of horizontal to vertical intercepts
Equivalent circle diameter	The diameter of the circle having the same area as the measured feature
Fullness Ratio	Shape factor proportional to the ratio of actual area to the circumscribed area

resin). More resin was poured on the sample to cover it. The cup was then placed in an oven (set at 80F°) and allowed to harden for 24 hours.

2) Thin sections 50 microns thick were made from the hardened resin block.

3) Black and white microscopic photographs were produced from the thin sections. The image analysis was performed on these photographs, Figure 4.7.

This technique was found to be relatively simple and to produce good, high contrast images. A scale was photographed in order to determine the calibration factor of the image analyzer.

Both field and feature measurements were performed on the micro-photographs. It was decided to measure the fiber density first and then determine the PSD according to Masounave et al (1980). The reader may refer to Chapter 2 for details of the method. Significant problems were encountered by this investigator when attempting to measure the fiber density. Problems included (Figure 4.7): 1) needle punching leaves many fibers in an inclined position, these were not cut during sectioning and therefore appear to be elongated; and 2) difficulties were encountered in separating very close fibers because they appear as a single unit during the count. Therefore, this procedure was not used and a one-dimensional (1-d) approach was proposed to determine 1-d pore parameters. An overview of the 1-d approach will be given next.

Figure 4.7. Enlarged photographic print of a thin section,
A4561

One-Dimensional Analysis

Referring to Figure 4.7, it is apparent that the pores, which are the white background, are highly connected and therefore can not be isolated for direct measurements. A one dimensional approach, which looks similar to the 1-d ANOSOL soil model (Ringorse-Voase 1990) was used to determine the pore parameters of the non-woven geotextiles. The steps needed to measure the one-dimensional parameters are:

- 1) The parts of interest (voids or fibers) were detected after performing a series of image transformations on the original image.
- 2) The detected binary image was stored in a binary plane.
- 3) A short QBasic program was written and used to divide the screen into a set of parallel horizontal lines spaced 4 pixels (equivalent to 5-10 microns) apart, and the divided screen was stored in another plane. The selection of the short spacing (4 pixels) was to make sure that fabric sections, or portion of them, which have a diameter of 30-50 microns, will be crossed by the lines.
- 4) A logical step (AND) was performed on the two planes (equivalent to intersection in set mathematics) and the results were stored in a third plane. This plane consisted of the horizontal intercepts in the pore space (Figure 4.8).
- 5) Measurements of the intercept lengths were then made.

The intercept length distribution (lp) for pores and (ls) for solids were measured using the feature measurement option. The

Figure 4.8. Illustration of the 1-d analysis

average pore intercept length (l_p), median pore intercept length (l_{p50}), the standard deviation (σ_l) and the number of measured or accepted intercepts (N_p) or (N_s) for pores and solids, respectively, were directly obtained using QUIC menu. More than 500 intercepts were measured for one image.

The pore sizes were assumed to be equal to the pore intercept lengths (l_p) and therefore the pore intercept length distribution will be the pore size distribution. The frequency and cumulative distributions (in increasing and decreasing order) were obtained by Quantiment 570, based on interval counts, i.e., the number of intercepts in a pore interval.

Test Results

Mercury Intrusion Porosimetry

Un-compressed Geotextiles

The data from mercury intrusion porosimetry tests were too numerous to be reported in a tabular format, therefore, they were presented in the form of cumulative and differential plots. The cumulative pore size distribution of the geotextiles are presented in Figures 4.9 to 4.13 for the 4 geotextile groups and T3341. The Figures plot the percentage of pore volume smaller than a certain pore size versus the pore diameter. Values of the total intrusion are given in Table 4.3. Each graph represents an average of two tests.

The differential pore size distributions are given in Figures

Figure 4.9: Pore size distribution of Polyfelt fabrics by MIP

Figure 4.10: Pore size distribution of Phillips fabrics, MIP

Figure 4.11: Pore size distribution of Treveira fabrics, MIP

Figure 4.12: Pore size distribution of Amoco fabrics, MIP

Figure 4.13: Pore size distribution of Tygar 3341, MIP

Table 4.3. Summary of MIP results of un-compressed geotextiles

Fabric	V_t ml/g	SSA m ² /g	O_{50} μm	O_{98} μm	O_{95} μm	O_{60} μm	O_{30} μm	O_{10} μm	O^* μm	n	O_{min}
TS600	5.83	0.16	152	316	248	163	141	114	145	0.84	32
TS700	5.28	0.15	164	307	254	175	150	110	146	0.83	25
TS800	5.19	0.13	179	333	272	192	161	120	155	0.83	36
TS1000	5.07	0.13	171	320	249	176	156	123	154	0.82	60
Ph4	4.07	0.14	124	335	278	138	98	68	113	0.79	42
Ph9	5.63	0.16	158	327	250	168	137	101	137	0.84	42
Ph12	5.95	0.17	159	306	237	166	137	102	139	0.84	40
TR1114	9.19	0.27	136	310	245	147	123	112	136	0.92	52
TR1125	8.39	0.22	151	313	256	155	139	129	152	0.92	47
TR1145	7.21	0.21	136	292	219	140	129	113	136	0.90	60
A4545	5.99	0.22	125	314	256	134	107	75	109	0.84	35
A4553	7.86	0.22	151	315	246	156	138	112	140	0.88	64
A4561	7.32	0.23	137	287	214	144	125	102	126	0.87	56
T3341	1.48	0.06	135	331	313	146	98	48	94	0.57	10

4.14 to 4.18 for the 4 geotextile groups and T3341. A differential plot shows the relative changes in the intruded pore volume between successive pore sizes. The plots are all in arithmetic scale.

A summary of the mercury intrusion results is given in Table 4.3. The Table gives values of the average, median, maximum (O_{98}), O_{95} and minimum pore sizes. The Table also includes O_{10} , O_{30} , O_{60} , the porosity (n) and specific surface area (SSA).

Compressed Geotextiles

The cumulative pore size distributions of 3 compressed geotextiles (TS800, Ph12, and A4553) are shown in Figures 4.19, 4.20 and Figure 4.21 respectively. The differential plots for the same geotextiles are shown in Figure 4.22 to 4.24.

The results of the compressibility tests, i.e., thickness versus stress and stress versus strain are shown in Figure 4.25 and Figure 4.26, respectively.

Results of the Image Analysis Tests

SEM Photographs

The SEM photographs used for the analysis of representative geotextiles from each group are shown in Appendix B (Figures B1 to B4). The photographs have different enlargements. As previously pointed out, the SEM photographs were found unsuitable for the determination of PSD of the non-woven geotextiles because of the depth effect, which is a characteristic of SEM. However, an attempt

Figure 4.14: Differential pore size distribution of Polyfelt fabrics, MIP

Figure 4.15: Differential pore size distribution of Phillips fabrics, MIP

Figure 4.16: Differential pore size distribution of Treveira fabrics, MIP

Figure 4.17: Differential pore size distribution of Amoco fabrics, MIP

Figure 4.18: Differential pore size distribution of Typar 3341, MIP

Figure 4.19: Pore size distribution of compressed TS800, MIP

Figure 4.20: Pore size distribution of compressed Ph12, MIP

Figure 4.21: Pore size distribution of compressed A4553, MIP

Figure 4.22: Differential pore size distribution of compressed TS800, MIP

Figure 4.23: Differential pore size distribution of compressed Ph12, MIP

Figure 4.24: Differential pore size distribution of compressed A4553, MIP

Figure 4.25: Compressibility of the geotextiles

Figure 4.26: Stress-strain relationship of the geotextiles

was made to determine some shape parameters from the SEM photographs. The shape parameters were assumed to be less affected by the subdivision of pores by the inner fabric layers. Table 4.4 shows the results of the shape parameters of the detected pores, measured by the QUIC menu of Quantimet 570. The parameters included are anisotropy, porosity, equivalent circle diameter, breadth, width, roundness and fullness ratio. The average ($\bar{x}$), median (x_{50}), and coefficient of variation (V_x) of each parameter is given in the Table. No measurements were made for Amoco fabric due to the poor contrast between the fibers and voids for this specific fabric.

Thin Sections

The photographs of the polished thin cross sections used to determine the 1-d parameters are shown in Appendix B (Figures B5 to B15). The distribution of intercept lines smaller than 410 microns (the pore size corresponding to the filling pressure of MIP) were considered to represent the pore size distribution of the geotextiles. The intercepts greater than 410 (the pore diameter corresponding to the filling pressure of MIP) were rejected because they represent lines that pass between fiber sections.

The distribution of fiber sections was not uniform for most of the fabrics (see Appendix B). This was due to the effect of needle punching and surface treatments. For some fibers (Amoco and Treveira) the needle locations can be clearly observed as they were represented by an open area surrounded by inclined fibers.

Table 4.4. Summary of the results from SEM photographs

		TS600		Ph9		TR1114			T3341
		D ¹	W	D	N ²	D	N	W ³	W
count		39	208	196	737	253	101	518	327
An ⁴		0.99	0.78	1.18	1.07	1.31	0.98	1.14	1.18
n ⁵		0.05	0.5	0.29	0.31	0.64	0.71	0.63	0.26
ESD ⁶	x ⁻	48	140	76	94	104	113	146	90
	x ₅₀	45	124	66	80	97	95	132	75
	V _x %	58	65	67	60	54	88	64	68
B ⁷	x ⁻	39	104	61	77	80	92	108	71
	x ₅₀	33	88	50	62	73	68	113	55
	V _x %	59	68	67	63	56	96	75	75
W ⁸	x ⁻	64	218	91	113	118	170	179	103
	x ₅₀	62	174	69	96	112	118	149	82
	V _x %	61	77	79	70	59	94	75	72
R ⁹	x ⁻	2.06	2.23	1.75	1.75	1.99	2.45	2.05	1.76
	x ₅₀	1.83	1.94	1.51	1.55	1.79	1.95	1.74	1.63
	V _x %	33	47	39	39	39	57	47	30
FR ¹⁰	x ⁻	0.81	0.76	0.81	0.80	0.78	0.78	0.80	0.80
	x ₅₀	0.82	0.78	0.82	0.82	0.79	0.79	0.81	0.82
	V _x	9	12	10	9	10	10	10	9
mean chord		36	131	59	72	66	93	101	72
fiber d (μm)		43		33		21			49

¹at a dense location²at a needle location³a wide field⁴anisotropy⁵porosity⁶equivalent circle⁷breadth⁸width⁹roundness¹⁰fullness ratio

Measurements were performed at more than one location for these fabrics and the average is taken as the overall pore size distribution. Measurements were performed on one (wide) field for fabrics which showed uniformity.

The Quantiment 570 histogram menu gives the cumulative and differential pore size distribution based on feature count, e.g., cumulative PSD is represented by the number (or percentage) of intercepts smaller or larger than l_p . This approach was found to give more emphasis to the smaller pores because all the pore sizes were equally weighted. Another approach was used to determine the pore size distribution based on pore lengths. The summation of pore lengths were calculated for equal pore length intervals and the cumulative PSD was obtained based on the percentage of pore lengths, i.e, the summation of pore lengths smaller than a certain pore length divided by the total summation of pore lengths

$$\%less \text{ than} = \frac{\sum_{j=1}^j l_{pj}}{\sum_{j=1}^t l_{pj}} \quad (4.5)$$

where (t) is the total number of pore intercepts.

Figures 4.27 to 4.30 show the cumulative pore size distributions of Polyfelt, Phillips, Treveira and Amoco fabrics as obtained using 1-d analysis. The distributions were given based on the two approaches, i.e, intercept count and summation of intercept lengths. A summary of the results is given in Table 4.5.

Figure 4.27: Pore size distribution of Polyfelt fabrics by 1-d image analysis

Figure 4.28: Pore size distribution of Philips fabrics
by 1-d image analysis

Figure 4.29: Pore size distribution of Treveira fabrics by 1-d image analysis

Figure 4.30: Pore size distribution of Amoco fabrics
by 1-d image analysis

Table 4.5. Summary of the results of 1-d analysis of thin sections

Fabric	lp	lp ₅₀	O ₉₅	V _x
TS700	161	140	346	63
TS800	171	156	372	62
TS1000	171	148	368	60
Ph4	149	117	354	69
Ph9	166	136	372	83
Ph12	184	163	372	57
TR1114	183	158	372	62
TR1125	182	172	372	60
TR1145	163	139	346	60
A4545	125	98	321	90
A4553	162	142	359	65
A4561	163	140	346	62

Discussion of the Results

Two advanced methods, MIP and image analysis, were used to determine the pore parameters of 14 non-woven geotextiles collected from 4 major geotextile manufacturers. However, due to the absence of standard procedures for the above tests, for this specific application, an effort was made by this investigator to utilize the equipment for the maximum benefit. In this section, the two methods will be assessed and their results will be discussed and compared with the most commonly used methods, i.e., AOS and FOS.

Mercury Intrusion Porosimetry

The major advantages of MIP, when used to determine the pore parameters of geotextiles, are its simplicity and speed. The method does not need any form of sample preparation, as the fabrics can be used without being dried. A typical test will be performed in less than an hour with the Autopore II system. Another advantage of the method is that the pore size distribution is obtainable for compressed non-woven geotextiles.

The cumulative pore size distribution of the un-compressed geotextiles were given in Figures 4.9 to 4.13. The geotextiles from the same manufacturer were plotted on the same figure. It can be observed from the plots that the pore size range of all the geotextiles is about the same (i.e., between 50 to 350 micron). The distribution of pore sizes is similar for geotextiles from the same manufacturer. However, the thin geotextiles have smaller pores compared to the relatively thicker ones from the same group (e.g.

TS600, Ph4 and A4545 compared to the thicker ones from the same group). In general, as determined by mercury intrusion porosimetry, thickness does not seem to have an influence on the pore size distribution for fabrics from the same manufacturer. This observation may seem surprising when one thinks of PSD according to the definition set by sieving methods (AOS and FOS). The latter are filtration tests which are expected to be very much influenced by the fabric thickness.

The differential pore size plots were given in Figures 4.14 to 4.18. These plots help to visually detect and compare the range of pores sizes of geotextiles. Figure 4.16 shows a very narrow (uniform) pore size range for Treveira fabrics compared to the other geotextile groups which have wider distributions. The sample of T3341 (Figure 4.13), which is the thinnest of all the fabrics, has the largest proportion of small pores (i.e., less than 75 microns) compared to the other fabrics.

The maximum and minimum pore sizes were obtained from MIP results (see Table 4.3). It is observed from the cumulative and differential plots that there is negligible intrusion between the filling pressure and the first pressure step, an indication that the pressure increase was not sufficient for intrusion. In the next steps controlled volume were injected into the fabric and pressure readings were recorded (controlled volume test). The injection continued until the pore volume left in the sample became smaller than the volume increment. The final pressure in the control pressure table was 20 psi (corresponds to a pore diameter of about 10 microns), but in most cases the pore volume between 50 microns

(corresponds to about 4 psi) and 10 microns was found to be less than the controlled volume increment. This indicated that the pressure reading preceding the final pressure would correspond to the minimum pore size. The maximum pore size is assumed to be equal to O_{98} which is the pore size that corresponds to 98% on the vertical scale of the cumulative plots. The pore sizes between O_{100} and O_{98} are assumed to represent the roughness of the geotextile.

A complete summary of the MIP test results was given in Table 4.3. This table shows that Treveira fabrics have the largest volume of pores per unit weight (V_i in ml/gm), therefore, the highest porosities compared to the other fabrics. Amoco fabrics have the second largest total intrusion volume while polyfelt and phillips fabrics are next in order. Analysis of variance was performed to compare V_i , n and SSA data (means), Table 4.3, for fabrics from different groups. The results of the statistical analysis shows significant differences, in these parameters, for geotextiles from different groups (see Table 4.6).

The cumulative pore size distributions for compressed TS800, Ph12 and A4553 were given in Figures 4.19, 4.20 and 4.21, respectively each for 0%, 15%, 30 and 50% strain. The corresponding differential plots were given in Figures 4.22, 4.23, and 4.24 for the same fabrics. As expected, compressibility affected both pore volumes and sizes. As the strain increased the pore volumes and sizes decreased. However, the response depended on the type of fabric. Amoco fabric experienced the largest decrease in pore volumes and sizes as a result of compression, while TS800 showed the least. Most of the decrease in pore volumes and sizes for TS800

Table 4.6. Results of the analysis of variance¹

parameter	geotxtile	mean	Duncan grouping ²
porosity	Polyfelt	0.830	C1
	Phillips	0.823	C1
	Treveira	0.913	A1
	Amoco	0.863	B1
V_t (ml/g)	Polyfelt	5.343	B2
	Phillips	5.217	B2
	Treveira	8.263	A2
	Amoco	7.057	A2
SSA (m ² /g)	Polyfelt	0.143	B3
	Phillips	0.157	B3
	Treveira	0.233	A3
	Amoco	0.223	A3

¹ confidence level is 0.95

² means with the same letter are not significantly different

and Ph12 took place during the first strain increment (0 to 15%), while A4553 experienced decrease in pore volume and sizes even for the last strain increment.

The response of the same fabrics to loading was given in Figures 4.25 and 4.26. The Figures revealed that A4553 is the most compressible of the three fabrics. Very small load may be needed to compress this fabric to 50% strain (30 Kpa) compared to above 100 Kpa for Ph12 and around 200 Kpa for TS800. Therefore, the compressibility characteristics of geotextiles are highly dependent on the manufacturing process.

Image Analysis

SEM Photographs

The SEM photographs were used to measure the surface porosity and pore shape parameters for representative geotextiles from each group. The major problems with the SEM photographs were the depth and shading effects, which made accurate detection of the surface pores difficult. Therefore, the images were considered unsuitable for the determination of pore size distribution.

A summary of the surface analysis of the geotextiles (TS600, Ph9, TR1114, and T3341) was given in Table 4.4. From visual inspection of the SEM photographs (Appendix B, Figures B1 to B5), Polyfelt and Typar fibers (TS600 and T3341) are elongated and straight, while Ph9, TR1114, and A4545 fibers are curled. The surface fibers of Ph9 are dense and some of them fused together, probably due to the heat treatment. The measured surface porosity

of this geotextile was 0.3, a value well below the surface porosities of Treveira and Amoco fabrics (0.7 and 0.5, respectively). Typar3341, although very thin, had the lowest surface porosity. The SEM scans to a shallow depth, therefore, the surface porosity values are an indication of the upper fiber layers spacing, and is not equivalent to the overall porosity of the fabric.

The other shape parameters showed small variation from fabric to fabric. Amongst the three fabric groups, the lowest breadth (the minimum feret) and width were measured for Phillips geotextile. However the maximum width, which is a measure of the elongation of pores, was measured for Polyfelt geotextile. The roundness and fullness values indicated that the surface pores have irregular shapes.

Thin Sections

Much can be learned from the visual inspection of the enlarged photographs of the thin sections (Appendix B; Figure B5 to B15). The photographs show rounded and elongated fibers. The elongated fibers are more frequent around the needle punch locations. The fiber sections are randomly distributed and in some cases very closely spaced (in the form of bundles). Closely spaced fiber sections were known as clogging sites or sites which trap soil particles when the fabric is used as a filter (Mlynarek et al, 1990). Such sites are more frequent in Polyfelt and Amoco fabrics (Appendix B), but less frequent in Phillips and Treveira fabrics.

Masounave et al (1980) used a probabilistic model to determine the PSD of thick non-woven fabrics (see Chapter 2). The model assumed that the fiber sections are circular in shape, randomly distributed and each fiber section is surrounded by voids from all sides. Referring to the photographs of the thin sections, and to the preceding discussion, the applicability of the model, for these images, becomes questionable. All the fiber sections are not circular and often exist in agglomerations or bundles which are not separable for analysis.

The pore size distribution was considered to be the same as the distribution of the intercepts of closely spaced horizontal lines with the pore space (1-d analysis). The pore size distribution was determined based on the number of intercepts (count) and on the intercept lengths (Figures 4.27 to 4.30). Because the horizontal lines are closely spaced, some of the horizontal lines pass between the fiber sections, and intercept lengths greater than 1000 microns were measured. A limiting maximum intercept length (the pore diameter equivalent to the filling pressure in MIP) was used.

The 1-d PSD is almost the same for fabrics from the same manufacturer (group). The thin fabrics, A4545 and Ph4 have higher percentage of small pores compared to their group members. Small pores dominated when the PSD was determined based on intercept counts, while large pores dominated when the PSD was determined based on pore lengths (Figures 4.27 to 4.30). The cumulative plots, based on count, are steep compared to the ones based on lengths. The minimum measured intercept lengths range from 25 to 38 microns

for all the fabrics.

A summary of the computer output of the 1-d analysis was given in Table 4.5. The average pore size from 1-d analysis ($\bar{p}$), the pore sizes O_{50} , O_{95} , and the coefficient of variation of pore sizes were given in the Table. It can be seen in the Table that the pore sizes are not uniform, since the coefficient of variation is greater than 50% for all the fabrics. The highest coefficients of variation were measured for the thin fabrics (A4545, Ph4 and TR1114). Thin fabrics, therefore, have wider range of pores compared to thicker fabrics.

Evaluation and Comparison of the Methods

Two advanced methods, MIP and image analysis were used to determine the pore parameters of 14 non-woven geotextiles obtained from four major manufacturers. Both methods were capable of measuring the pore size distribution of the geotextiles. The MIP was found to be simpler and faster due to the following: 1) no sample preparation was necessary; 2) with the Autopore II system the test was computer controlled and all that was needed were a few input parameters; and 3) the equipment used is commercially available. However, the geotextiles could not be directly analyzed by the image analyzer, therefore, a series of sample preparation steps were needed in order to obtain a high contrast image. The sample preparation required special chemicals, expertise in making thin sections and special microscopes for producing enlarged photographs.

The volume control MIP test option was preferred over the pressure table option as it automatically produced closely spaced test points. However for the image analysis the thin sections were preferred over the SEM photographs. The depth effect made the latter unsuitable for pore size determinations.

Table 4.7 shows the median O_{50} , average O' and O_{95} values as obtained by MIP and 1-d image analysis. Also in the Table are the AOS and FOS values (from the literature). It is apparent from the Table that the O_{95} value obtained from MIP and image analysis are much higher than AOS and FOS values (corresponding to O_{95} as obtained from sieving). The FOS values are the lowest, probably because graded material (glass beads or sand) are used for the FOS test. Anyhow, there is no logic in comparing FOS and AOS with MIP and image analysis because the former are filtration tests which are highly dependent on the surface porosity, thickness and other structural properties of the geotextile, whereas, the latter are a function of the pore spaces and pore volume.

The results of MIP and image analysis both indicated similar pore size distribution of geotextiles from the same manufacturer. The major difference between geotextiles from the same group is the thickness. However there is significant difference in porosity, specific surface area and surface porosity between different geotextile groups. These differences are the ones responsible for the difference in their filtration behavior.

Table 4.7. Comparisons of the pore size parameters

fabric	1-d image analysis		MIP		dry sieving	wet sieving
	lp_{50}	lp_{95}	O_{50}	O_{95}	AOS	FOS
TS700	140	346	164	254	125-180	83
TS800	156	372	179	272	106-150	-
TS1000	148	368	171	249	<75	-
Ph4	117	354	124	278	150-212	105
Ph9	136	372	158	250	150-212	100
Ph12	163	372	159	237	125-180	117
TR1114	158	372	136	245	150-212	195
TR1125	172	372	151	256	125-212	103
TR1145	139	346	136	219	106-150	85
A4545	98	321	121	256	<212	-
A4553	142	359	151	246	<212	-
A4561	140	346	137	214	<212	-

CHAPTER 5: MODELING OF THE RETENTION CAPACITY OF NON-WOVEN GEOTEXTILES

Introduction

The basic objective of a filter in drainage applications is to retain the filtered material while allowing free flow of water. The behavior of a soil/geotextile system depends on the properties of the geotextile and the protected soil and on the flow conditions. The conditions under which a soil particle leaves the soil and finds its way through the fabric filter or is trapped within it depend on: 1) the ability of the soil to retain its own fines, i.e., the suffusion characteristics (internal stability) of the soil; 2) the geometrical conditions of the fabric; and 3) the flow conditions, i.e., the seepage velocities and hydraulic gradients.

The existing filtration models for granular filters assume the filter performance to be basically controlled by the structural properties of the filter media (Silveira, 1965 and Wittmann, 1979). Variations in the hydraulic conditions are neglected and it is assumed that the seepage velocity is sufficient to move the unstable soil particles.

An attempt is made in this chapter to improve Silveira's model to fit soil/non-woven geotextile systems. The model developed is based on the overall pore size distribution of the geotextile and considers the effect of other structural properties such as thickness and porosity. Considerations are given to the internal

stability of the base soil, i.e., its ability to retain its own fines. The results of testing programs carried out by the this investigator and other investigators to check the model are presented. A retention criterion based on the model will be presented.

Basic Considerations

The structural properties of non-woven geotextile filters can be of either micro or macro nature. The micro structure of non-woven geotextiles consists of short fibers or filaments arranged in all directions and bonded together into a planar structure. The filaments or short fibers are first arranged into a loose web, then bonded together mechanically, chemically or thermally. A net of pores is created by the arrangement of fibers. Currently, for most practical situations, the fabric is expressed by a representative pore size which is the apparent opening size (AOS) or O_{95} , which means a pore size larger than 95% of all the pores. Alternately, the fabric may be expressed by O_{50} or the median pore size.

Important macro-structural properties of the fabric are thickness and porosity. Thickness affects the tortuosity, which reflects the path or distance a soil particle will travel in order to cross the fabric. Although very important, the thickness is neglected in the current filter models.

The protected soil is represented by a characteristic grain size, usually d_{85} and/or d_{50} and d_{15} , while the other grain sizes and their frequencies are neglected. No direct consideration is given

to the internal stability of the protected soil.

Most of the existing filter criteria for geotextiles are empirical and were derived from laboratory tests. In these criteria a characteristic base soil size, d_x , is compared to a characteristic geotextile opening size, O_x , by means of a piping ratio P_r , where:

$$P_r > \frac{O_x}{d_x} \quad (5.1)$$

These criteria give no consideration to the overall pore size distribution of the fabric filter. Since both the fabric and the protected soil are characterized by parameters having a wide range of variation, statistical treatment of the filtration phenomenon will lead to improved predictions. It is important to have a good measure of the pore size distribution (PSD) of the geotextile. With the PSD known, a probabilistic model expressing the geometric hindrance of the movement of base grains through voids of the filter, similar to those developed for granular filters, can be developed (Silveira, 1965 and Wittmann, 1979). Accurate determinations of the pore size distribution of non-woven geotextiles have been made by this investigator using the mercury intrusion porosimetry and image analysis. The results were reported in the previous chapter. The mercury intrusion porosimetry results will be the basis for this model although pore size determinations by image analysis may be used.

The problem is approached in two ways: first absorbing Markov Chains are used to illustrate the model and to estimate the

expected number of confrontations needed to retain a soil particle with a known diameter. This is followed by a reliability analysis in order to give a more general understanding of the ability of a certain geotextile to retain a soil particle. The model is extended to evaluate the effect of compressibility on the retention capacity of geotextiles. Illustrative examples are given to demonstrate the model. The model is extended to evaluate the effect of non-woven geotextile compressibility on its retention ability.

The Model

To evaluate the retention ability of the geotextile filter, the following assumptions are made:

- 1) The flow drag is significant enough to move soil particles adjacent to the geotextile.
- 2) Soil units are spherical.
- 3) The geotextiles openings are circular.

The retention criteria become a geometric problem, i.e., depending on the geometrical characteristics of the filter media, a function of the pore size distribution of the geotextile and the movable grain size of the protected soil that is to be filtered.

The fabric is considered as a set of (m) parallel thin layers (sieves), each having the same pore size distribution and with pores randomly distributed, Figure 5.1. The solid part of the fabric is distributed between the pores. The value of (m) depends on the filter length (thickness) and the height of an individual confrontation. This height is assumed to be equal to the largest

Figure 5.1. Schematic diagram of the model

pore size $O_{\max}$, in order for a confrontation to contain the largest particle that can practically intrude it, Figure 5.1. The value of $O_{\max}$ can be approximated by O_{98} which is the pore size greater than 98% of the fabric pores, and should be obtained using mercury intrusion porosimetry (refer to the previous chapter).

For a given geotextile with a known pore size distribution (see Figure 5.2 for the PSD of a typical non-woven geotextile) any particle with a grain size greater than the maximum pore, $O_{\max}$, of the geotextile will be retained. A particle smaller than the smallest pore size of the geotextile filter, $O_{\min}$, will have a high probability of being able to escape or pipe. The probability of a particle with a diameter d , where $O_{\min} < d < O_{\max}$, passing the filter fabric or being retained within it depends on the size of the particle, the pore size distribution of the fabric and its solid content (porosity) and the fabric thickness.

Analysis Using Absorbing Markov Chain

The average distance traveled by a particle having a diameter (d), where $O_{\min} < d < O_{\max}$, before being absorbed or caught by the fabric, can be estimated using the Absorbing Markov Chain, if the absorbing and non-absorbing states can be defined. There are three transition states to be considered, one non-absorbing state and two absorbing states. All the pores with diameters greater than (d) are non-absorbing states. The absorbing states include the solid part of the material ($1-n$ in Figure 5.2) and the cumulative frequency of the pores smaller than the size of the particle to be retained

Figure 5.2: Pore size distribution of a typical geotextile

(d). To obtain the frequency of each transitional state, the pore volumes are normalized to the total volume of the piece of geotextile that was intruded. This is done by multiplying the ordinates of the pore size distribution curve by the porosity (the dashed line in Figure 5.2). For a particle having a diameter (d) where $O_{\min} < d < O_{\max}$, the non absorbing states will have a frequency of $(n-p^*)$, whereas the absorbing states have a frequency $(1-n)$ for the solid part and p^* for the pore sizes less than (d). The quantity (p^*) is equal to $(n \text{ times } p)$. The probability matrix for Absorbing Markov Chains can be written as:

$$P = \begin{array}{c|ccc} & \begin{matrix} 1 & 2 & 3 \end{matrix} \\ \hline \begin{matrix} 1 \\ 2 \\ 3 \end{matrix} & \begin{matrix} n-np & np & 1-n \end{matrix} \\ & \begin{matrix} 0 & 1 & 0 \\ 0 & 0 & 1 \end{matrix} \end{array}$$

The upper row and left column designate the three transitional states. Number 1 is a non-absorbing state while 2 and 3 are absorbing states. To determine the expected number of times or confrontations any of the above states may be occupied before absorption, a procedure originally suggested by Kemeny and Snell (1960) and described by Harr (1977) is followed. The matrix is put in canonical form:

$$P = \begin{array}{cc} I_r & Q_{rs} \\ T & Q \end{array}$$

Where (r) and (s) refer to the absorbing and non-absorbing states respectively (r is equal to 2 and s to 1). The value of I_r is a

unit matrix of order (r), Q_{rs} is a null matrix with (r) rows and (s) columns, T is an (s times r) matrix and Q is a square matrix of order (s).

The probability matrix in its canonical form is;

$$P = \begin{bmatrix} - & 2 & 3 & 1 \\ 2 & 1 & 0 & 0 \\ 3 & 0 & 1 & 0 \\ 1 & np & 1-n & n-np \end{bmatrix}$$

Let:

$$M = [I_s - Q]^{-1} \quad (5.2)$$

Here, I_s is a unit matrix of order (s) (here s equal to 1). The elements of matrix M define the expected number of confrontations a particle will travel before being absorbed.

Therefore, for this case, the expected number of confrontations (m) a particle with a diameter (d), where $0_{\min} < d < 0_{\max}$, will pass before being absorbed is equal to

$$m^- = \frac{1}{1-n(1-p)} \quad (5.3)$$

Where (p) is the percentage of pores smaller than the particle size (d) and is obtained from the pore size distribution curve of the fabric; (n) is the porosity.

If the porosity and the PSD of a certain geotextile fabric are known, the expected number of confrontations, m^- , and hence the average fabric thickness needed to stabilize a certain soil

particle with a diameter (d), where $O_{\min} < d < O_{\max}$, can be estimated. The average thickness is equal to m multiplied by O_{98} (the maximum pore size of the non-woven geotextile).

For particles with a diameter less than $O_{\min}$, $p=0$ and the average number of confrontations (m) is:

$$m = \frac{1}{(1-p)} \quad (5.4)$$

Reliability Analysis

The above solution gives the average number of confrontations needed to retain a soil particle with a diameter (d), where $d < O_{\max}$. A reliability analysis is needed in order to select or determine upper limits for the number of confrontations, (m), and hence design values.

The failure of a confrontation can be described as its inability to retain a certain soil particle, and the failure probability according to the model is $(1-R)$. The system will be considered to have failed to satisfy the retention criterion for a certain soil particle if and only if all the confrontations fail to retain the soil particle, i.e., it is a parallel system. The reliability of a parallel system with m confrontations is, Harr (1987):

$$R = 1 - \prod_{i=1}^{i=m} (1-R_i) \quad (5.5)$$

R_i is the reliability of confrontation i and is

equal to $(1-n+np)$ for all confrontations. Therefore,

$$R = 1 - (n - np)^m \quad (5.6)$$

By rearranging Equation (5.5), m may be obtained as:

$$m = \frac{\log(1-R)}{\log(n-np)} \quad (5.7)$$

For $p=0$;

$$m = \frac{\log(1-R)}{\log(n)} \quad (5.8)$$

The design value of (m) is, therefore, a function of the acceptable reliability, porosity and pore size distribution of the geotextile and the size of the particle to be filtered. Figures 5.3 to 5.6 show the estimated (m) values for different (n) and R values. As shown in the Figures, the number of confrontation needed to retain a soil particle increases as (p) decreases. The increase in (m) for small particles ($p < 15\%$) is sharper as the porosity increases. Therefore, according to the model, the retention of small soil particles is basically porosity dependent. Knowing the pore size distribution, porosity of the geotextile used and the size of particle to be retained, the number of confrontations needed to retain a certain soil particle and hence the fabric thickness can be estimated from Figures 5.3 to 5.6. The required geotextile thickness, for a given reliability, R , is obtained by multiplying (m) by O_{98} which is the maximum pore size of the

Figure 5.3: The number of confrontations needed to stabilize a particle, $n=0.80$

Figure 5.4: The number of confrontations needed to stabilize a particle, $n=0.85$

Figure 5.5: The number of confrontations needed to stabilize a particle, $n=0.90$

Figure 5.6: The number of confrontations needed to stabilize a particle, $n=0.95$

geotextile.

The effect of (m) on the reliability of geotextile filters when filtering fine grained soils ($d < O_{min}$) is shown in Figure 5.7. This Figure also emphasizes the effect of porosity on the retention ability. Figure 5.7 shows that by increasing the fabric thickness its reliability increases. However, the rate by which R improves is a function of the porosity of the fabric, i.e., using thicker fabric for a relatively porous filter (e.g., $n = 0.95$) will not add much to its reliability. A typical example to illustrate the model will be given.

Example

- The PSD of a typical non-woven geotextile is given in Figure 5.2. If the porosity of the geotextile is equal to 0.90, determine:
- 1) the expected number of confrontations and the average geotextile thickness required to trap a soil particle with $d < O_{min}$, $d=75$ microns and $d=150$ microns; and
 - 2) the reliability in of a 2.5 mm geotextile when retaining the particles given in 1)?

Solution

From the PSD curve in Figure 5.2, $p=16\%$ and 68% for $d=75$ and $d=150$ microns, respectively. From Equations 5.3 and 5.4, the expected number of confrontations needed, m , to retain $d < O_{min}$, $d=75$ and $d=150$ microns are 10, 4 and 2, respectively. Since O_{98} is equal to 330 microns (from Figure 5.2), the corresponding average

Figure 5.7: The effect of fabric thickness on reliability for $p=0$

thicknesses are 3.3 mm for $d < O_{\min}$, 1.7 mm for $d = 75$ microns and 0.70 mm for $d = 150$ microns.

For the second part of the example, a thickness of 2.5 mm implies that m is equal to 8. From Equation (5.6) the calculated reliability values are: 57%, when $d < O_{\min}$; 89%, when $d = 75$ microns; and 99.9%, when $d = 150$ microns.

The above example shows that the model estimates are consistent and reasonable.

Compressibility Effects

Non-woven geotextiles are known to be compressible under loads. The compressibility affects both the porosity and the pore size and volume distribution and thus the filtration performance. A major advantage of mercury intrusion porosimetry, as mentioned in the previous chapter, is that the pore size distribution of compressed geotextiles can be easily measured. The developed model can be extended to evaluate the effect of geotextile (non-woven) compressibility on the retention ability and reliability. Compressibility is usually given in terms of strain. However, if a stress strain relationship is available for a certain geotextile, then the effect of loading can be readily assessed.

When a geotextile is compressed to a strain ϵ , then

$$\epsilon = \frac{\delta e}{(1+e)} \quad (5.9)$$

Where (e) is the non-compressed void ratio and δe is the

decrease in void ratio due to compression. The value of ϵ can also be related to the porosity (n) by:

$$\epsilon = \delta e(1-n) \quad (5.10)$$

The porosity (n_c) of a compressed fabric is

$$n_c = \frac{e_c}{(1+e_c)} \quad (5.11)$$

When the fabric is compressed, the compressed void ratio, (e_c) is equal to $(e-\delta e)$. From Equations (5.10) and (5.11)

$$n_c = \frac{(n-\epsilon)}{(1-\epsilon)} \quad (5.12)$$

From Equation (5.6), the reliability of a compressed geotextile is

$$R = 1 - (n_c - n_c p_c)^{m_c} \quad (5.13)$$

The subscript (c) stands for compression. Assuming one dimensional compression, (m_c) can be approximated by $m(1-\epsilon)$ and Equation (5.13) can be expressed by:

$$R = 1 - \left[\frac{(n-\epsilon)}{(1-\epsilon)} (1-p_c) \right]^{m(1-\epsilon)} \quad (5.14)$$

For $p_c=0$

$$R = 1 - \left[\frac{(n-\epsilon)}{(1-\epsilon)} \right]^{m(1-\epsilon)} \quad (5.15)$$

Therefore, in order to evaluate the reliability of a compressed geotextile in retaining a soil particle, the pore size distribution of the compressed geotextile under the specified strain level ϵ needs to be known (m and n are properties of the uncompressed fabric). A load deformation relationship is needed for the fabric in order to determine the reliability for a known surcharge load.

There are two physical changes that a geotextile undergoes during compression: the first is a decrease in its pore volumes and sizes and therefore improve its retention ability, while the second is a decrease in thickness or in the number of confrontations needed for retention and this tend to lower the reliability. Therefore, in order to have a feeling about the effect of both phenomena, an example will be presented.

Example

Consider the geotextiles A4553, TS800 and Ph12 for which the compressed pore size distributions are known (Chapter 4). Find their reliabilities when retaining soil particles with $d < O_{\min}$, $d=50$, 75, 100 and 150 microns for the strain levels 0, 15, 30 and 50%.

Solution

To use Equation (5.14), p_c values for the pore sizes corresponding to the particle diameters must be known. The values can be obtained from the Figures 4.19, 4.20 and 4.21, from the previous chapter, for TS800, A4553 and Ph12, respectively. The

information about the thickness and porosity of the given fabrics are obtained from the Tables 4.1 and 4.3. Reliability values are computed for each geotextile for the particle diameter and the strain levels, and the results are plotted in Figures 5.8 , 5.9, and 5.10. The Figures show the effect of compressibility on reliability for A4553, TS800 and Ph12, respectively. It is apparent from the Figures that compressibility improves the retention ability of these fabrics. However, the improvement is a function of the strain level, the size of particle to be retained and the type of fabric.

Most of the improvement for TS800 and Ph12 was observed for the first strain level ($\epsilon=15\%$) and little improvement is observed for 30 and 50% strain levels. However, for A4553 considerable improvement is observed for the first and last strain increments, i.e., 0-15% and 30-50% (Figure 5.8). Regarding the effect of particle size, the Figures indicate little improvement for $d < O_{\min}$ and $d=250$ microns, i.e., for very small and large soil particles. The best improvement is for the sizes 100 to 150 microns which correspond to the average or most probable pore sizes, Table 4.3. It is of interest to know the effect of compressibility on the retention ability of a specific fabric to fine soil particles ($d < O_{\min}$). Figure 5.11 shows a plot of reliability versus compressibility for a hypothetical geotextile with $n=0.9$ and $m=10$. The plot gives no effect of compressibility on the retention ability of geotextiles for fine soil particles ($d < O_{\min}$).

Typical stress-strain envelopes, Figure 4.26, show that a very small surcharge is needed to compress the fabrics. Less than 15 Kpa

Figure 5.8: Effect of compressibility on the retention capacity of A4553

Figure 5.9: Effect of compressibility on the retention capacity of TS800

Figure 5.10: Effect of compressibility on the retention capacity of Ph12

Figure 5.11: Compressibility effects on the reliability of a geotextile
 $p=0$, $m=10$ and $n=0.9$

(equivalent to 0.75 meters of fill) is needed to compress the three fabrics to 15% strain. Loads needed to compress the fabrics to higher strain levels depend on the type of geotextile. Therefore, with very small surcharge loads considerable improvement is expected in the retention ability of fabrics when retaining particles with $O_{\min} < d < O_{\max}$. In summary:

- 1) compressibility has a minor effect on the retention ability of geotextiles to fine soil particles ($d < O_{\min}$);
- 2) there is considerable improvement for $O_{\min} < d < O_{\max}$ and the degree of improvement is dependent on the compressibility characteristics of the geotextile, the type of fabric and the size of particle to be retained;
- 3) best improvements are expected for sizes very close to the median pore size of the geotextile; and
- 4) very small surcharges may be needed to obtain the benefits from compression.

Considerations for Variability in Base Soil Gradation

The above model has the limitation of considering a single grain size to be retained, whereas in reality the protected soil is formed of various grain sizes. The effect of the base soil gradation on its filterability was emphasized by many researchers (Sherard, 1984; Lafleur, 1984; and Kenney and Lau 1985). It was found that for granular filters, filter criteria used for base soils that are internally stable will not apply for internally unstable (gap graded and some broadly graded) soils. Another

important finding reported by Sherard (1984) is that for well graded base soils, a granular filter sufficiently fine to catch the d_{85} size of a stable soil will also catch the finer base particles. Therefore, for this application, the following assumptions will be made based on previous experiences with granular filters:

- 1) The model applies only for internally stable soils
- 2) For internally stable soils, the geotextile will satisfy the retention criteria if it can retain the d_{85} size for a desired confidence or reliability.

A procedure is needed to determine whether a base soil is internally stable or not.

The Internal Stability of Base Soils

The internal stability of a cohesionless material results from its ability to prevent the loss of its own small particles due to seepage forces. Suffusion is the transport of small particles from a soil or within its structure. Kovacs (1981) presented an extensive review on the work done on this topic. Different methods were suggested by Kovacs (1981) and elsewhere (Kenney and Lau, 1985) to assess the internal stability of soils. A consistent and rational approach to evaluate the potential instability of base soils will be presented. The approach also facilitates the determination of a critical particle size (d_{cr}) which is the largest particle size that is able to move within the soil mass. It is an extension to the procedure suggested by Kezdi (1969) and based on

Terzaghi's filter criteria. The procedure is improved to take care of the inherent conservancy of Terzaghi's criteria.

According to the original model suggested by Kezdi (1969), the grain size distribution of a base soil is divided at an arbitrary point d_n into two parts, coarse and fine (Figure 5.12). The coarse skeleton is regarded as the filter for the remaining fine particles. The Terzaghi criterion gives

$$4d_{85g} > d_{15f} > 4d_{15g} \quad (5.16)$$

The numbers stand for the percentage smaller than size (d) and the subscripts (f) and (s) stand for filter and soil respectively. If S_n is the percentage smaller than d_n on the grain size distribution curve then Terzaghi's criterion can be given by the following equation

$$4d_{.85S_n} > d_{.85S_n+15} > 4d_{.15S_n} \quad (5.17)$$

The first inequality determines the retention criteria, while the second inequality determines the permeability criteria. Only the first inequality is of interest. The value of $4d_{.85S_n}$ and $d_{.85S_n+15}$ are plotted against d_n (Figure 5.12). The critical diameter, d_{cr} , is at the intersection point of $4d_{.85S_n}$ and $d_{.85S_n+15}$. The soil grains being smaller than this size are able to move through the large pores. If the two curves have no point of intersection and the $4d_{.85S_n}$ curve is always above the $d_{.85S_n+15}$ curve, no suffusion is expected.

Lafleur et al (1989) and Sherarad et al (1984) have shown that the original Terzaghi's criterion with a retention ratio of 4 to 5

Figure 5.12. Application of Kezdi's method to determine the self filtering ability of a soil (after Kovacs, 1981)

is conservative and involves a factor of safety of approximately 2. Sherard et al (1984) showed that there is a very narrow margin between filter failure and success, well defined by d_{15f}/d_{85} equal to 9 (factor of safety=1). Therefore, for this study, and in order to avoid any conservancy, a retention ratio of 9 will be used to evaluate the internal stability of cohesionless base soils. The following inequality will be more appropriate.

$$9d_{.85Sn} > d_{.85Sn+15} \quad (5.18)$$

Knowing the grain size distribution of a base soil one can go through its grain size distribution curve, plot the retention inequality, Equation (5.18), and check its stability. If unstable, a critical diameter, characterizing the maximum grain size able to move within the soil mass, can be defined. The results will serve as a basis to assess the stability of base soils for the model.

Verification of the model Predictions

In this section, results of laboratory filtration tests carried out by the author and obtained from the literature will be presented in order to verify the model predictions. The tests performed by this writer are given the name "slurry tests", because the base soil was in slurry form. The test results obtained from the literature will be named after the respective first author. A description of the laboratory tests will be given first, followed by the test results and discussions. The discussion will focus on checking the general trends of the model and defining a critical

acceptable reliability that can be used as a retention criterion. Comparisons of the suggested retention criterion will be made with those from the literature.

The Slurry Tests

Two series of tests were performed. In the first series a clayey silt slurry was used to test the retention capacity of 4 non-woven needle punched fabrics (TS600, TS700, TS800 and TS1000) manufactured by Polyfelt and 3 needle punched heat treated fabrics (Ph4, Ph9 and Ph12) manufactured by Phillips Fibers. Information about the structural properties of these fabrics were given in the previous chapter. The structural properties of interest are summarized in Table 5.1.

The testing program for the first series included running retention tests in which the retained material was a 50 gram per liter natural clayey silt slurry. The tests were carried out in a large filtration device similar to a conventional gradient ratio apparatus. A thick slurry was first formed by mixing 400 gram of oven dried soil (clayey silt passing No. 200 sieve) with 400 ml of water. The tested fabric was installed and allowed to stay in water overnight. Before testing, the apparatus was filled with water to a specified height (8 liters of water). Water was stirred for one minute using a stirrer (rotating blade) and the prepared slurry was added. After all the slurry was added (90 seconds) the mix was stirred for 30 seconds. The bottom valve of the filtration device was immediately opened to drain the slurry. The drained water was collected in large containers where fines were allowed to settle.

Table 5.1. The structural properties of the fabrics used for the slurry tests

geote- xtile	intrusion ml/gram	O ₅₀ (μ m)	O ₉₈ (μ m)	thickness mm	n	min pore	FOS
TS600	5.827	152	316	1.5	0.84	32	95
TS700	5.282	164	320	2.6	0.83	25	83
TS800	5.191	179	333	2.95	0.83	36	
TS1000	5.067	171	320	4.3	0.82	60	
Ph4	4.07	124	335	1.016	0.79	42	117
Ph9	5.38	158	327	2.3	0.84	42	100
Ph12	5.948	159	320	3.62	0.84	40	104

The soil was separated using siphoning. The entrained material was then oven dried to obtain its weight.

The second series of the slurry tests involved only two geotextiles, (i.e., TS600 and Ph9). The geotextiles were tested with different gradations of Ottawa sand in order to determine the effect of particle size and on the retention capacity of the types of fabrics. Only 100 grams of soil was used in order to avoid sedimentation of the sand during stirring. The soil gradations and properties are given in Table 5.2. The testing procedures is similar to those of the first series. The entrained material was collected and clarified water removed by siphoning. The weight of the oven dried soil was obtained.

Faure et al, (1986)^b

Faure et al, (1986)^b performed long term filtration tests using 6 loose base soils and 6 fabrics. The gradation curves of the soils are given in Figure 5.13. The properties of the non-woven geotextiles are given in Table 5.3. Of interest are the properties of the first three fabrics manufactured by polyfelt (TS500, TS600, TS700) because information is available about their structural properties. The apparatus used is similar to the EPM filtration device shown in Figure 2.23. A 3.5 cm thick loose soil sample was used under a hydraulic gradient of 10. Long term filtration tests were run for each of the geotextiles with all the soils. The filtrate was collected on a filter paper for each test, and the filter paper was weighed every 24 hours to measure the amount of

Table 5.2. Gradation of the soils used in series(2)
of the slurry tests

soil designation	range of gradation (U.S sieves)	average grain size (μm)
1	-#200	less than 75
2	+#200 to -#140	106
3	+#140 to -#100	150
4	+#100 to -#70	212

Table 5.3. The properteis of the geotextiles used
by Faure, et al (1986)

fabric	mass/unit area (g/m_2)	thickness (mm)	FOS (μm)
TS500	180	1.9	115
TS600	220	2.2	95
TS700	295	2.7	83
C34	250	2.35	110
N4	175	1.3	100
NP 153	173	1.8	95

N.B.

M_5 , M_6 , and M_7 are monoclase soils obtained by sieving between two successive sieves.

Figure 5.13. Properties of soils used by Faure et al, 1986

entrained soil.

Qureshi, (1990)

Qureshi (1990), presented results of an extensive investigation on the long term filtration behavior of non-woven geotextiles when tested with gap graded soils. The retention tests were run at Syracuse University, Syracuse, N.Y. Six non-woven geotextiles were involved (i.e., TR1114, TR1125, TR1145, Ph4, Ph8 and Ph12) with three gap graded silts. The TR-fabrics are needle punched while Ph-fabrics are heat treated. The structural properties of the fabrics are given in Table 5.4. The pore size distribution of these fabrics have been measured (Chapter 4). Three gap graded sandy silts were used in the retention tests. The grain size distribution of the soils (SIL CO SIL 75, SIL CO SIL 106 and SIL CO SIL 250) are given in Figure 5.14. Qureshi used a type of apparatus similar to that used by Faure et al, (1986)^b. A quantity of 240 grams of loose silt, in slurry form, was used. Water and fine materials coming out of the system were collected for the first 24 hours of the test and put through a centrifuge to separate water from fines.

Discussion of the Test Results

The Slurry Tests

Table 5.5 shows the test results given in terms of the amount and percentage of entrained soil. The reliability of the fabrics was calculated using Equation (5.6), Table 5.5.

Table 5.4. The structural properties of the geotextiles tested by Qureshi, (1990)

geotextile	mass/unit area oz/yd ²	AOS (microns)	FOS (microns)	thickness (mm)
TR1114	4.2	210	195	1.65
TR1125	7.4	210	103	2.79
TR1145	13.5	-	-	4.44
Ph4	4	180	105	1.016
Ph8	8	210	100	2.03
Ph12	12	210	117	3.05

Table 5.5. The test results for the first series of slurry tests

geotextile	m	R	entrained soil (gm)
TS600	5	58.2	386.4
TS700	8	77.5	374
TS800	9	79.2	353.4
TS1000	13	92.4	344.9
Ph4	3	50	373.8
Ph9	7	72.9	357.6
Ph12	11	85.3	353.7

Figure 5.14. Grain size distribution of SIL CO SIL soils (after Qureshi, 1990)

The results show that for the same type of fabric (needle punched or heat treated) the amount of the entrained material decreases as the reliability increases, Figure 5.15. However when the response of the two fabrics is compared, it can be shown that the amount of entrained material is affected by the manufacturing process. For example, TS800 and Ph4 allowed the same amount of soil to pass while their reliabilities were different (85% versus 50%). The previous investigation of the pore parameters of non-woven geotextiles showed that needle punched/heat treated geotextiles have low surface porosity compared to the needle punched (untreated). This low surface porosity is probably responsible for the improvement in the retention ability of heat treated geotextiles. The amount of entrained silt is more than expected and this is probably due to the viscous nature of the slurries and the excitation and turbulence caused by stirring.

Table 5.6 shows the results of the second series of slurry tests where fine Ottawa sand was used. The reliability of each fabric is computed, Equation (5.6). The average grain size is used in the calculations due to the narrow gradation of the sands. The results are plotted against the average particle size, Figure 5.16. Although there is no substantial difference between the two fabrics (TS600 and Ph9) in terms of thickness and porosity, the heat treated fabric, Ph9, had a better retention capacity compared to the needle punched, TS600. This is due to its lower surface porosity (see Table 4.4). The retention ability of the two fabrics increases sharply as the average grain size of the retained sand becomes larger. The heat treated fabrics showed better than

Figure 5.15: The entrained soil versus reliability (first series)

Table 5.6. The test results for the second series of the slurry tests

sand gradation	TS600		Ph9	
	entrained (gm)	R	entrained (gm)	R
-#200	85.08	58.2	8.59	72.9
#200 - #140	53.63	67.6	0.36	83.7
#140 - #100	19.03	84.5	0	96.4
#100 - #70	3.46	99.99	0	99.99

Figure 5.16: The entrained soil versus reliability (second series)

expected retention ability, while the needle punched showed good agreement with the model.

In spite of the cited differences, the model showed consistency in predicting the response of a specific fabric when retaining dilute slurries.

Faure et al, (1986)^b

A summary of the results of the tests run by Faure et al, (1986)^b is given in Figure 5.17. The amounts of entrained soil were determined from Figure 5.17, for TS-fabrics (Table 5.7). Before calculating the reliability of TS-fabrics, the internal stability of the soils was evaluated using the procedure suggested in this chapter. All the soils were found to be internally stable, therefore their d_{85} values will be used for reliability calculations. Table 5.8, shows the calculated reliability values for TS500, TS600, and TS700. The structural properties of the fabrics were obtained from Table 5.3 and Figure 4.9. Since all TS-fabrics have very similar pore size distributions, Figure 4.9, their average pore size distribution is adopted for TS500.

The amount of the entrained soil (g/m_2) was plotted against the reliability for each data point, Figure 5.18. A consistent and sharp decrease in the amount of entrained material is observed as the reliability increases. As the reliability approaches 80%, a very small amount of entrained material was measured.

Faure et al, (1986)^b presented a retention criterion based on their test results. A filter system would be stable if FOS/d_{85}

Figure 5.17. Mass of soil entrained during 24 hours versus FOS/d_{85} (after Faure et al 1986)

Table 5.7. Test results (g/m^2) as deduced from Figure 5.17
(Faure et al, 1986)¹

soil#	2	3	4	5	6	7
TS500	118.65 (2.1)	37.8 (1.05)	1675 (2.17)	635 (1.65)	112.9 (1.15)	60.24 (.809)
TS600	(1.86)	(0.86)	945.6 (1.87)	237.25 (1.36)	23.6 (0.95)	(0.68)
TS700	11.45 (1.63)	(0.75)	145 (1.60)	57.3 (1.189)	13.3 (0.83)	(0.59)

¹ values in () are FOS/d_{85}

Table 5.8: The reliability of TS-fabrics tested
by Faure et al, (1986)

soil#	d_{85} (μm)	TS500		TS600		TS700	
		p%	R%	p%	R%	p%	R%
2	51	2	68.9	2	74.4	3	82.3
3	110	10	81.3	10	85.9	10	90.3
4	53	2	68.9	2	74.4	3	82.3
5	70	3	70.7	3	76.2	3	82.3
6	100	7	77.3	7	82.2	7	87.4
7	140	30	95.9	35	98.6	25	97.7

Figure 5.18: Entrained soil versus reliability (Faure et al, 1986)

ratio lies between 1-2.

A plot of FOS/d_{85} versus reliability is made in order to compare the two criteria, Figure 5.19. The value of FOS/d_{85} consistently decreases as R increases. A reliability of 80% will correspond to a FOS/d_{85} value of about 1.5, which is the same average value suggested by Faure et al, (1986)^b. This implies a good agreement between the two approaches.

Qureshi, (1990)

The results of the retention tests performed by Qureshi are shown in Table 5.9. As for the previous discussion the soils (SIL CO SIL) were first checked for internal stability using the procedure presented in this chapter. SIL CO SIL 75 and SIL CO SIL 106 were found to be internally stable while SIL CO SIL 250 was found to be unstable. Only stable soils were accepted for the verification of the model predictions. The calculated reliability values for the fabrics tested by Qureshi are shown in Table 5.10 for all the silts. The Ph8 fabric was assumed to have the same pore size distribution as Ph9 since they have almost the same thickness and no significant variation was observed between the pore size distribution of Phillips fabrics.

The amount of entrained material was plotted against reliability, for all the fabrics and soil combinations, excluding SIL CO SIL 250 which is internally unstable (Figure 5.20). For both fabrics the Figure shows that as the reliability increases the amount of entrained material sharply decreases. Small amounts of

Figure 5.19: FOS/d85 versus reliability (Faure et al, 1986)

Table 5.9. Results of the retention tests performed
by Qureshi, (1990)

geotextile	SIL CO SIL 75	SIL CO SIL 106	SIL CO SIL 250
	entrained (gms)	entrained (gms)	entrained (gms)
TR1145	0.50	-	-
TR1125	1.35	0.17	0.32
TR1114	8.22	0.31	0.33
Ph12	0.12	-	-
Ph8	0.22	1.46	0.38
Ph4	1.87	0.83	0.35

Figure 5.10. The reliability of the fabrics tested by
Qureshi, (1990)

fabric	m	SIL CO SIL 75		SIL CO SIL 106		SIL CO SIL 250	
		p	R%	p	R%	p	R%
TR1145	13	0	74.5	2	80.5	14	96.4
TR1125	8	0	48.7	2	56.3	6	68.7
TR1114	5	0	34.1	3	43.4	14	69.0
Ph4	3	1	52.2	24	78.3	47	92.7
Ph9	6	1	66.9	7	77.3	20	90.8
Ph12	9	1	81	7	89.2	20	97.2

Figure 5.20: Entrained soil versus reliability (Qureshi , 1990)

entrained material was collected for a reliability higher than 80%. A plot of FOS/d₈₅ versus reliability shows a consistent decrease in FOS/d₈₅ as the reliability increases (Figure 5.21). A similar trend was observed when the same plot was made for the results of Faure et al, (1986)^b (Figure 5.19). A reliability of 80% will correspond to FOS/d₈₅ of about 2, in Figure 5.21. The suggested retention criterion by Qureshi was FOS/d₈₅>3.0. Qureshi justified the selection of this very high limit by the fact that his criterion was developed for gap graded soils, therefore, higher opening sizes are needed to guard against clogging and blinding.

The above analysis shows a good agreement between the model predictions and results of retention tests from the literature. It is important to mention that only limited good retention data have been found in the literature. The writer was able to find only these two complete sets of data. The analysis of the results also shows that a reliability of 80% will always assure minimum piping through the geotextile.

Figure 5.22 shows a plot of the entrained soil versus reliability but with SIL CO SIL 250 included to assess the effect of accepting internally unstable soils. The squared points are for the internally unstable soil. It can be observed from the Figure (the square symbols on the graph) and from Table 5.9 that the amount of entrained material for SIL CO SIL 250 is approximately constant for all the fabrics and was not affected by the differences in the structural properties of the fabrics. This implies that unstable soils behave differently when filtered by geotextiles. This observation supports the assumption that the

Figure 5.21: FOS/d85 versus reliability (Qureshi, 1990)

Figure 5.22: Entrained soil versus reliability (Qureshi, 1990)

proposed model is not suitable for, and should not be applied to, potentially unstable soils.

Summary

A probabilistic model to evaluate the retention capacity of non-woven geotextiles was presented in this chapter. The model assesses the effect of pore size distribution, porosity thickness and compressibility of a fabric on its ability to retain soil particles. To apply the model the overall pore size distribution of the geotextile should be known. This writer suggests that the mercury intrusion porosimetry method be used to determine the pore size distribution of the geotextile filter because of its simplicity and acceptable accuracy.

In order to consider the effect of base soil gradation, it was assumed, based on previous experiences with granular filters, that the model applies only for internally stable soils. A simple approach to assess the internal stability of base soils, based on the analysis of their grain size distribution, was presented for use with the model.

The model was validated with laboratory data and good agreement was obtained between the computed reliability values and the amount of entrained materials. A critical value of reliability (80%) was determined, above which the stability of the base soil is insured (only minor losses will be experienced). The proposed retention model has the following advantages:

- 1) it is based on a rational development;
- 2) it considers the important structural properties of the

geotextiles, i.e., the pore size distribution, thickness and porosity;

3) it considers the internal stability of the base soil; and

4) it is a reliability based approach and is therefore amenable to risk analysis

CHAPTER 6: LONG TERM PERFORMANCE OF NON-WOVEN GEOTEXTILES

Introduction

Geotextiles installed in earth structures can be altered with time by many phenomena such as clogging, blocking and piping. These phenomena are a function of the structural properties of the geotextile and the soil adjacent to it. The geotextile structure is currently described by mass per unit area, AOS or FOS, and thickness; whereas, the soil structure is described by characteristic grain sizes such as d_{15} , d_{50} , and d_{85} , and the uniformity coefficient.

A comprehensive review of the different aspects related to the long term performance of the soil/geotextile systems was given in Chapter (2). This chapter focuses on the influence of the geotextile and soil structure on the long term properties of soil/geotextile systems. A brief review of the current approaches used to analyze long term filtration data will be given. Next a comprehensive set of long term filtration data provided by Qureshi, (1990) will be analyzed and discussed. Morphological evaluation (by the image analyzer) of the long term performance of field samples of geotextiles obtained from exhumed highway prefabricated drains will also be presented. The drains were installed 8, 6 and 4 years ago.

Current Approaches

There are three major approaches used to evaluate long term performance of geotextiles: the gradient ratio test; long term filtration tests; and morphological evaluation of field and laboratory samples.

A quantitative parameter, the gradient ratio, is obtained from the gradient ratio test. A gradient ratio greater than 3 will indicate clogging. The limitations of the gradient ratio test were given, in detail, in Chapter (2).

Long term filtration tests are continued until no significant change in the flow rate of the system is observed, i.e., the system stabilizes. Koerner and Ku, (1982) suggested that a system stabilizes when the final slope of the filtration curve becomes very small (refer to Figure 2.22). A relatively large slope will indicate clogging and blinding of the system. No threshold value for a critical condition was given by Koerner and Ku. Mlynarek et al, (1991) used the shape of the long term filtration curve to evaluate long term filtration tests (refer to Figure 2.25). They found that a geotextile will satisfy the long term performance requirements as long as its filtration behavior curve is of type I or III of Figure 2.19.

Mlynarek et al, (1991) defined a long term property termed flux decay, defined by the ratio of seepage velocity at 20 hours to seepage velocity at 2 hours after the start of the test. The flux decay measures the rate of decrease of flow during the initial stages of the test. A good relationship between flux decay and d_{10}

of the soils was found.

The morphological approach involves analysis of magnified images (microscopic photographs) of geotextile samples from in-service drains or from laboratory simulation tests, using direct observations or image analysis techniques.

Long Term Filtration Tests by Qureshi, (1990)

Qureshi, (1990) presented a comprehensive data on long term filtration tests performed to study the behavior of gap graded soils. The retention tests described in the previous chapter are part of these tests. Six geotextiles (Ph4, Ph8, Ph12, TR1114, TR1125 and TR1145) were tested with Ottawa sand, three cohesionless silts (SIL CO SIL 75, SIL CO SIL and SIL CO SIL 250) and their mixtures with sand (20, 40, 60 and 80% silts), i.e., a total of 16 soils. The grain size distributions of these soils are shown in Figure 6.1. A hydraulic gradient of 10 was established during the test. The pore parameters of the above geotextiles were measured by this investigator and were given in Chapter (4).

The apparatus and procedure of the filtration tests were described in Chapter (5). Typical long term filtration data produced by Qureshi are given in Figure 6.2.

The results of the long term tests of Ottawa sand and the 20% mixtures (SIL CO SIL soils with Ottawa sand) were extracted from graphs similar to Figure 6.2. The filtration data have been replotted for each soil (Figure 6.3 to 6.6) in order to assess the effect of the geotextile structure on the long term performance.

Figure 6.1. The grain size distributions of the soils used by Qureshi, 1990

Figure 6.2. Typical results of the long term filtration tests performed by Qureshi, 1990

Figure 6.3. Plots of filtration data for Ottawa Sand/fabric systems

Figure 6.4. Plots of filtration data for SIL CO SIL 75/fabric systems

Figure 6.5. Plots of Filtration data for SIL CO SIL 106/fabric systems

Figure 6.6. Plots of filtration data for SIL CO SIL 250/fabric systems

Proposed Approaches for Analysis

Long term filtration tests are usually carried out with gap graded soils in order to speed up the test, and for the creation of unfavorable (conservative) testing conditions. With these soils the decrease of flow is expected to be a function of the internal soil movement and the geotextile structure. This analysis is an attempt to assess the effect of the soil gradation and the geotextile structure on the long term performance of soil/geotextile systems.

Two approaches will be used to analyze the data. In the first, the effect of the geotextile and soil structures on the flux decay of the system will be studied. An attempt will be made to relate the flux decay of gap graded soil systems to the suffusion characteristics (internal stability) of the soil using concepts suggested in the previous chapter. In the second approach a qualitative assessment will be made for the long term behavior of different geotextiles when tested with gap graded soils. The objective is to study the effect of fabric structure on the long term performance of soil geotextile systems.

Flux Decay

The only quantitative long term parameter that is obtainable from the data provided by Qureshi, (1990) is the flux decay. In order to assess the effects of the soil and geotextile structure on long term parameters an attempt will be made to: 1) relate flux decay to the suffusion properties of the soil; and 2) investigate the effect of geotextile structure on flux decay.

The soil mixtures used by Qureshi (1990) can be described as gap graded cohesionless soils (Figure 6.1). The internal stability of the soils was evaluated using the approach proposed in Chapter (5). The only soils found to be internally stable are SIL CO SIL 75, SIL CO SIL 106 and the 60 and 80% mixtures of SIL CO SIL 75 with sand. The severity of the gap gradation differs from one soil mixture to the other and generally increases as the percentage of silt decreases. The piping ratio, defined by the maximum value of $d_{0.85s_n+15}$ divided by $d_{0.85s_n}$ (refer to Chapter 5), can be considered as a measure of the severity of the gap gradation. The higher the piping ratio the more gap graded the soil is and the faster small particles can move within a soil skeleton. Table 6.1 shows the computed piping ratios of the silty soils and their corresponding flux decays.

Qureshi, (1990) attempted to relate flux decay to characteristic grain sizes of the soils, i.e., d_{10} , d_{60} and d_{85} , but was unsuccessful especially for the unstable soils (SIL CO SIL 106 and 250). This is probably because the behavior of unstable soils is more influenced by the degree of gap gradation.

A plot of the flux decay values versus piping ratio, for all fabric and soil combinations, is given in Figure 6.7. All flux decay values greater than unity were excluded because they represent unstable conditions of soil/geotextile systems. In order to exclude the effects of geotextile structure on flux decay, separate plots are given for Phillips and Treveira fabrics in Figure 6.8 and 6.9, respectively. The results of a non-parametric correlation performed using Spearman's Model show no significant

Table 6.1. Piping ratio and flux decay values, obtained from Qureshi, 1990

soil		piping rato	geotextile			
SIL CO SIL	% silt		TR1114	TR1125	Ph4	Ph8
75	20	20.00	0.77	0.69	+	-
	40	12.40	0.70	-	+	0.81
	60	8.30	0.85	0.35	+	0.92
	80	6.60	0.88	0.52	+	0.87
	100	5.90	0.85	-	+	1.02
106	20	25.70	0.83	0.78	0.73	0.72
	40	14.00	0.85	0.90	0.87	1.06
	60	10.70	1.01	0.83	0.86	0.88
	80	9.00	0.67	0.77	1.31	0.80
	100	8.00	0.97	0.76	0.84	0.94
250	20	26.00	0.80	0.70	0.69	0.69
	40	20.30	0.82	0.46	0.81	1.01
	60	10.70	0.82	0.56	1.11	0.81
	80	11.90	0.81	0.48	0.73	0.88
	100	9.70	0.95	0.72	1.01	-

- not available

+ could not be obtained

Figure 6.7. The effect of piping ratio on flux decay for all soils

Figure 6.8. Variation of flux decay with piping ratio for Phillips fabrics

Figure 6.9. Variation of flux decay with piping ratio for Treveira fabrics

correlation between flux decay and piping ratio of the unstable soils. Correlation coefficients (r_s) of -0.64, -0.81, -0.26, and -0.11, and corresponding probabilities of 0.053, 0.170, 0.50, and 0.78 were obtained for Ph4, Ph8, TR1114 and TR1125, respectively.

The flux decay values given in Table 6.1 indicate several cases of instability of the soil geotextile systems (flux decay values greater than unity). The instability was observed for the heat treated fabrics. This observation indicates inefficiency of parameters obtained during the initial stages of a long term filtration test (e.g., flux decay and gradient ratio), especially for heat treated fabric/systems. To assess the effect of geotextile structure on the flux decay of unstable-soil/geotextile systems, analysis of variance was carried out to compare the flux decay values (the means) for Ph4, Ph8, TR1114 and TR1125 systems, using Duncan model. The analysis showed no significant difference between the means for Phillips fabrics and TR1114. However, significant difference between the flux decays of TR1125 and the other geotextile systems was obtained.

The above analysis showed that, for unstable soil/geotextile systems, neither the piping ratio nor the geotextile structure seem to have significant influence on the flux decay of soil/geotextile systems.

Long Term Performance and Fabric Structure

The results of the long term filtration tests of the soil/geotextile systems were plotted in Figures 6.3 to 6.6. The

plots were given in terms of permeability versus time for four soils (Ottawa sand and the 20% mixtures of SIL CO SIL 75, 106 and 250 with Ottawa sand, respectively). The following observations can be made from the Figures.

- 1) For all tests the soil/geotextile systems did not seem to have reached complete stabilization even when two or more close readings were obtained before the tests were stopped. Such close readings were obtained far before test completion but subsequent decrease in flow rate was experienced.

- 2) The filtration data of the geotextiles when tested with Ottawa sand were plotted in Figure 6.3. The Figure indicates higher permeability values when thick fabrics were used compared to thin fabrics from the same group. For example Ph12 data plot above Ph8 data, for most of the points, and both plot above Ph4 data. For Treveira fabrics TR1145, data plot above TR1125 data for most of the points. The only exception is TR1114, which plots above all the fabrics although it is the thinnest of all Treveira fabrics. The low permeability values of thin fabric systems are attributed to the fact that thin fabrics are more permeable than thick fabrics, therefore, can drain water faster. This produces higher flow rates, accelerates soil densification, and reduces the permeability of the sand/fabric system. However, this writer is unable to explain the unusual behavior of TR1114.

- 3) The unstable soils (Figures 6.4 to 6.6) show filtration behavior similar to Ottawa sand. The thin fabric/soil systems have lower permeability when compared to thick fabric/soil

systems for fabrics from the same manufacturer. This is because thin fabrics allow higher flow rates, causing faster movement of the small particles in the soil structure. This will definitely accelerate the growth of the filter cake on the fabric surface and lead to low permeability. However, if considerable piping occurs, the cake build up will be delayed, and the permeability of the fabric system will be higher during piping. In conclusion: the fabric structure plays an important role in the formation of the filter cake. A fabric with high permeability (porosity is high and/or the fabric is thin) accelerates the evacuation of water from the soil, and high flow rates will be induced within the soil structure which accelerate the filter cake formation. A fabric with low permeability (porosity is low and/or fabric is thick) reduces the rate of water flow from the soil and therefore delays the formation of the filter cake.

4) An early increase in the permeability of the soil/fabric system was experienced when Phillips fabrics, which are heat treated, were tested with both stable (Ottawa sand) and unstable (SIL CO SIL mixtures). The increase was followed by a decrease in permeability. It is interesting to observe that all Phillips fabrics behaved the same way. It can be observed from the Figures 6.4 and 6.5 that the slope of the points preceding the increase in permeability is about the same as the slope of the points following the increase. The increase in permeability was probably caused by a partial collapse of the filter cake which indicates a fabric's self-wash ability.

Giroud (1982) observed the same phenomena for heat bonded non-woven geotextiles and attributed it to the smooth geotextile surface. According to Giroud, soil particles in contact with the smooth geotextile surface are rather free to move about and, therefore, have a tendency to form a loose structure. This writer attributes the self-wash behavior to the low surface porosities of heat bonded geotextiles (low surface porosity was measured by the image analyzer for Phillips fabrics, Table 4.4). The low surface porosity causes faster cake formation and increased gradients on the cake which results in its partial collapse.

The above discussion reveals that the long term behavior of soil/geotextile systems is influenced by both the geotextile and soil structure. Stable and unstable soils showed similar trends for their long term behavior. For Ottawa sand the decrease in flow was attributed to the densification of the sand. The rate of densification and therefore decrease in flow rate was controlled by the fabric structure. However, for unstable soils the decrease of flow rate was attributed to the internal soil movement (suffusion). Suffusion causes cake build up and reduction of the porosity of the fabric. Suffusion is enhanced by high flow rates, while, the cake build up is influenced by the retention ability of the fabric. Both are affected by the geotextile structure.

Morphological Evaluation

The results of a morphological evaluation of the long term

performance of geotextile filters obtained from exhumed highway edge drains will be given in this section. All the exhumed drains were manufactured by the Monsanto company and with Amoco 4545 geotextile filter. The objective is to determine the factors which affect the performance of geotextile filters used in highway subdrainage and to quantify their clogging level using image analysis.

Prefabricated drains were exhumed from 3 sites on I-65 in Indiana. The oldest drains were installed in 1984 (site 1), while the drains in site 2 and 3 were installed in 1986 and 1988 respectively. The subgrade soils in both sites were classified as CL. Details about the field investigation program were given in Chapter (3).

Before any morphological evaluation was made, the contaminated geotextile samples were treated in order to conserve the in-situ conditions. Contaminated samples (about 5 cm²) were cut from the exhumed drains in the direction facing the subbase, placed in small cans and allowed to dry. An impregnating material, Ultra Low Viscosity resin produced by Polysciences Inc. was then poured around the fabric until the fabric was completely immersed. The cans were then placed in an oven, set to 80°F, for at least 12 hours to solidify. A thin section was made from each geotextile sample and microscopic photographs were obtained for the thin sections.

Figures 6.10, 6.11 and 6.12 show microscopic photographs of portions of the thin sections obtained for geotextiles from site (1), site (2) and site (3), respectively. The small white points

Figure 6.10. Enlarged photographic prints of thin section of contaminated A4545, site (1)

Figure 6.11. Enlarged photographic prints of thin section of contaminated A4545, site (2)

Figure 6.12. Enlarged photographic prints of thin section of contaminated A4545, site (3)

represent fabric cross sections while the elongated objects represent pieces of fibers. The black color represents silt and clay particles, while, the blocky objects are sand particles. The upper portion of the photographs correspond to the part of the geotextile facing the pavement side.

Visual inspection of the photographs indicate some degree of particle entrapment in almost all the photographs. Minor entrapment is observed in Figure 6.12, i.e., geotextiles from site (3), whereas, more silt and sand particles were entrapped in geotextiles from site (1) and site (2). The geotextiles installed in 1986 (i.e., site 2), indicated the highest degree of clogging. The inspection indicated that silt and clay particles were entrapped around dense fiber locations (clogging sites). Sand particles are either found on the outer side of the geotextile, i.e., the side facing the pavement, or in the large pore spaces inside the fabric (Figure 6.11). The age of the geotextile seemed to have no effect on its clogging level.

The intensity of caking was found to be more to the inner side of the drain than to the outer side. This was observed visually after removing the drains, and is confirmed by the microscopic photographs (Figure 6.11). This phenomenon was caused by the inner fibers catching the silt and clay particles in suspension when water flowed inside the drain. It is expected that the formation of the inner cake will be influenced by the fabric roughness and the concentration of soil particles in suspension. This observation is contrary to the laboratory models used to simulate long term flow of highway subdrainage (refer to Figure 2.17 and Figure 2.19).

The porosities of the contaminated fabrics were measured by the image analyzer. The measured porosities were 0.79, 0.46 and 0.84 for geotextiles from site (1), site (2) and site (3) respectively. The porosity of the virgin fabric was 0.84 (Table 4.4). Therefore, complete clogging did not take place and many of the large pores were available to convey water into the drain. Another expectation is that some of the entrapped soils can still be removed by the water flowing across the fabric. However, this inside caking, if allowed to dry, may not be easily washed out by water passing across the fabric.

Some bulky soil particles (in the size of fine sand) can be seen in Figure 6.12, trapped inside the geotextile. Clogging is probable if a large quantity of relatively large soil particles, about 100 microns (fine sand), is trapped inside the geotextile. Such particles are more likely to stay permanently inside the drain. This can happen if pumping of fine sand occurs and the sand is carried by water to the geotextile. The latter observation agrees with previous findings by other researchers (Lennoz-Gratin, 1987; Rollin et al, 1982).

The above observations showed that: 1) morphological analysis is helpful in assessing the clogging of field geotextile samples and leads to better understanding of the factors and mechanisms influencing the long term performance, i.e., clogging and blinding in this case; 2) existence of clogging sites affects the long term performance of geotextiles installed in highway subdrains; 3) clogging may be anticipated if fine sand particles are carried across the fabric by water; and 4) blinding is possible, inside the

drain, if water carrying high concentration of suspended fines passes in the drain.

CHAPTER 7: SUMMARY AND CONCLUSIONS

This study evaluated prefabricated highway edge drains in order to set guidelines for their proper selection. Two primary evaluations received attention. The first was the effect of installation and field loading on the flow capacity of the drain, and the second was the performance of the geotextile filter. The conclusions of the research on the drain and on the fabric will be given separately. These conclusions were based on field investigation, laboratory testing and theoretical modeling.

Hydraulic and Strength Requirements

For a proper and acceptable performance of a PFHED, its core should satisfy the following hydraulic and strength requirements:

- 1) In order for a PFHED to perform as well or better than an efficient conventional drain, when draining dense subbases, it should have a minimum flow capacity of 15-18 gallons/ minute/ foot for longitudinal drain slopes of 2% and 1% respectively. This flow capacity should be obtained by ASTM-D4716-87.
- 2) In order for a PFHED to resist the induced compaction and in-service stresses it should have a minimum normal compressive strength of 8100 psf and an inclined compressive strength of 7500 psf at 80° inclination. The direct compressive stress may be obtained using the GRI test method.

3. Construction control of PFHEDs is necessary in order to minimize the potential construction related damages. Borescoping was suggested for the control and should be included in the construction specifications. It is believed that borescope evaluation will help to detect problems, on-site, and therefore suggest measures to improve construction procedures.

Filter Requirements of the Fabric

An intensive research was executed on the filtration characteristics of non-woven geotextiles. The following is a summary of the research findings:

1) Laboratory testing to measure the pore parameters of 14 nonwoven geotextiles obtained from different manufacturers was accomplished. Mercury intrusion porosimetry and image analysis were used. Due to absence of standard testing methods for geotextiles using these devices, special testing techniques were developed. The following observations were obtained:

i) Mercury porosimetry is superior to image analysis because it is faster, simpler, requires no sample preparation, and measures the PSD of compressed geotextiles.

ii) The PSD of fabrics obtained from the same manufacturer were found to be similar. The major difference between these fabrics was thickness.

iii) Significant differences in PSD, porosity and specific surface area were obtained for fabrics obtained

from different manufacturers.

iv) The pore sizes measured by MIP and 1-d image analysis are higher than the ones obtained by sieving techniques. This is because the latter are filtration tests and are influenced by thickness, porosity and surface conditions.

2) A probabilistic retention model was developed for the prediction of the reliability of nonwoven geotextiles. The model was based on PSD, porosity and thickness of the geotextile. The model was extended to include the effect of compressibility on the retention ability of geotextiles. It's predictions were evaluated by laboratory test data. The model has the following advantages over the existing empirical approaches:

- i) it is based on a rational approach;
- ii) it considers the important structural properties of the geotextile;
- iii) it considers the suffusion characteristics of the base soil;
- iv) it is a reliability-based approach, therefore, it gives the flexibility of choosing the reliability according to the severity of the problem and the importance of the project;

A procedure was given to check the internal stability of the base soil.

3) The long term performance of geotextiles was investigated by analyzing extensive long term filtration data provided by

Qureshi (1989). The effect of the geotextile structure and gap gradation of the soil on the long term performance was studied. The analysis showed no significant correlation between flux decay and either the geotextile structure or piping ratio.

4) The morphological evaluation of field geotextile samples showed partial clogging of the geotextiles. The age of the geotextiles did not seem to affect their long term performance as filters. Soil particles were found, trapped in the geotextiles, at locations of dense fabrics, i.e., clogging sites.

Recommendation for Future Research

The limited field investigation carried out by this writer revealed shortcomings for the type of drain used in Indiana. Guidelines were given to improve existing practice. However further research will be needed in order to assure the efficiency of the guidelines given in this report. The following areas need more research:

1) Better estimates of the compaction induced stresses on the drain are needed. The estimates made in this report assumed a rigid drain, but in reality drains are flexible. Therefore, the effect of the flexibility of the drain on the compaction induced stresses needs further study. There is a tendency towards using sand backfill in place of the excavated material. This will increase the cost of the drains, and a cost/benefit study is recommended before requiring such sand backfill.

- 2) Intensive field investigation of PFHEDs is needed in order to relate the pavement type, pavement distresses, subgrade material and subbase material to the performance of both the drainage core and geotextile filter.
- 3) This study provided valuable information on the microstructural properties of nonwoven geotextiles. Further study of the microstructure is needed, in order to understand the effects of needle punching and surface treatments on the filtration behavior of fabrics.
- 4) Further study of the long term performance of the geotextile filter is needed. It is recommended that morphological studies be performed on exhumed geotextile obtained from sites with different pavement, subgrade and subbase conditions, in order to relate the long term performance to the above variables.

LIST OF REFERENCES

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Ahmed, Z. (1992) Personal Communications

American Association of State Highway Transportation Officials, "Specifications for Geotextiles" Task Force No. 25 (ASHTO-ABC-ARBTA), (from Koerner, 1990).

American Society of Testing and Materials ,ASTM Designation No. D4716-87 "Constant Head Hydraulic Transmissivity (In-Plane Flow) of Geotextiles and Geotextile Related Products" , ASTM standards

American Society of Testing and Materials ,ASTM Designation No. D4833-88 "Index Puncture Resistance of Geotextiles, Geomembrane and Related Products", ASTM Standards

Baldwin, J. S. (1991) "Evaluation of Interstate 77 Pumping Pavement" Paper No. 910788 Submitted to the 70th Annual TRB Meeting, Washington D. C.

Baumgardner, R. H.; and Mathis, D. M. (1989) "Experimental Project 12, Concrete Pavement drainage Rehabilitation" Office of Highway Operations, Demonstration Projects Division, US Department of Transportation

Calhoun, C. O. (1972) "Development of Design Criterion and Acceptance Specifications for Plastic Filter Cloths" Technical Report No. S-72-7, US Army Corps of Engineers, Waterways Experimental Station, Vicksburg, Miss., 83 p.

Carrol, R. G., Jr. (1987) "Hydraulic Properties of Geotextiles" Geotextile Testing and the Design Engineer, ASTM, STP 952, pp. 7-20

Carrol, R. G., Jr. (1983) "Geotextile Filter Criteria" Transportation Research Record, TRR 916, pp. 46-53

Cedergren, H. R./Koa (1972) "Development of Guidelines for the Design of Subsurface Drainage Systems for Highway Pavement Structural Sections" Federal Highway Administration, Report No. FHWA-RD-72-30

Cedergren, H. R. (1988) "Why All Important Pavements Should be Well Drained" Effects of Geosynthetics on Soil Properties and on Environmental and Soil Systems, Transportation Research Record, TRR 1188, pp. 56-62

Christopher, B. R.; and Holtz, R. D. (1985) "Geotextile Engineering manual" U. S. Federal Highway Administration Report No. FHWA-TS-86/203 1044 p.

Converse, F. J. (1956) "Compaction of Cohesive Soil by Low-Frequency Vibration" ASTM Special publication No. 206, pp. 75-81.

Dempsey, R. M. (1989) "Hydraulic Requirements of Geocomposite Filter Materials Utilized in Pavement Subdrainage" Geotextiles and Geomembranes, 8, pp. 191-215

Faisal Haj Ali (1991) "The Influence of Filter jacket and Core geometry on Longitudinal Permeability of a Prefabricated Drain" Soils and Foundations, Vol. 31, No 3, pp. 120-126

Faure Y., Gourc J. P.; Millot F.; and Sunjoto, S. (1986)^a "Theoretical and Experimental Determination of the Filtration Opening size of Geotextiles" Proceedings of the Third International Conference on Geotextiles, Vienna pp. 1275-1280

Faure, Y.; Gourc, J. P.; Brochier, P.; and Rollin, A. L. (1986)^b "Soil Geotextile Interaction in Filter Systems" Proceeding of the Third International Conference on Geotextiles, Vienna pp. 1207-1212

Fischer, G. R.; Christopher, B. R.; and Holtz, R. D. (1990) "Filter Criteria Based on Pore Size Distribution" Proceeding of the Fourth International Conference on Geotextiles. pp. 289-294

Fleckenstein, I. J.; and Allen, D. L. (1991) "Evaluation and Performance of Geocomposite Edge Drains in Kentucky" Paper No. 910339 Submitted to the 70th Annual TRB Meeting, Jan. 1991, Washington D. C.

Frobel, R. K. (1991) "Eccentric (Angled) Loading of Prefabricated Highway Edge Drains" paper No. 910787, Submitted to the 70th Annual TRB Meeting, Jan. 1991, Washington D. C.

Garcia-Bengochea, I. (1978) "The Relation Between Permeability and Pore Size Distribution of Compacted Clayey Silts" Msc. Thesis, Purdue University, 179 p.

Geosynthetic Research Institute (GRI), Test Method GC4 (1989) "Compressive Behavior of Prefabricated Edge Drains and Sheet Drains" Geosynthetic Research Institute, Drexel University

Giroud J. P., (1982) "Filter Criteria for Geotextiles" Proceeding of the Second International Conference on Geotextiles, Las Vegas, NV, Vol. 1, pp. 103

Giroud, J. P. (1987) "Geotextiles and Related Products" Geotechnical Modelling and Applications, Editor Sayed M. Sayed, Gulf Publishing Company, pp. 355-390

- Grace, H. P. (1956) "Structure and Performance of Filter Media" Journal of the American Institute of Chemical Engineering, Vol 2, No. 3, pp. 307-315
- Harr, M. E. (1977) Mechanics of Particulate Media, a Probabilistic Approach, Mc-Graw Hill, 543 p.
- Harr, M. E. (1987) Reliability Based Design in Civil Engineering, Mc-Graw Hill, 290 p.
- Hoare, D. J. (1982) "Synthetic Fabrics as Soil Filters - A Review" Journal of the Geotechnical Engineering Division, ASCE, Vol. 108, GT10, pp. 1230-1245
- Hodek, R. J. (1972) "Mechanism for the compaction and response of Kaolinite", Ph.D Thesis, Purdue University, 267 p.
- Halburton, T. A.; and Wood, P. D. (1982) "Evaluation of the United States Corps of Engineer Gradient Ratio Test for Geotextiles" Proceeding of the Second International Conference on Geotextiles pp. 97-101
- Holtz, R. D.; and Kovacs, W. D. (1981) An Introduction to Geotechnical Engineering, Prentice Hall Inc., N.J., 733 p.
- Ingold, T. S. (1985) "A Theoretical and Laboratory Investigation of Alternating Flow Criteria for Woven Structures" Geotextiles and Geomembranes 2, pp. 31-45
- Janssen, D. J. (1982) "A Dynamic Test to Predict the Field Behavior of Filter Fabrics used in pavement Subdrains" Paper Prepared for the 62nd TRB Meeting, Washington D. C.
- Kenny, T.C.; and Lau, D. (1985), "Internal stability of Granular Soils" Canadian Geotechnical Journal, 21, pp. 634-643
- Kezdi, A. (1969) "Increase of Protective Capacity of Flood Control Dikes" (in Hungarian), Department of Geotechnique, Budapest Technical University, Report No. 1 (from Kovacs, 1981)
- Koerner, R. M.; and Hwu Bao-Lin (1989) "Geocomposite Drainage Systems, Highway Edge Drains and Retaining Wall Sheet Drains" Research Project # 88-15, Pennsylvania Department of Transportation
- Koerner, R. M.; and Hwu Bao-Lin (1991) "Designing and Testing of Prefabricated Highway Edge Drains" Paper Submitted to the 70th Annual TRB Meeting, Washington D. C.
- Koerner, R. M. (1990) Designing with Geosynthetics, Second Ed., Prentice Hall, N. J., 652 p.
- Koerner, R. M.; and Ko, F. K. (1982) "Laboratory Studies on Long Term Draining Capability of Geotextiles" Proceedings of the Second International Conference on Geotextiles pp. 91-95

Koerner, G. R.; and Koerner, R. M. (1992) "Tests to Determine How Highway Drainage Systems Perform Using Geosynthetics" Geotechnical Fabric Report, March 1992, pp. 18-28

Kovacs, G. (1981) Seepage Hydraulics, Elsevier Scientific Publishing Company, 730 p., pp. 349-363

Kraemer, S. R.; and Smith, A. D. (1986) "Geocomposite Drains, Vol.1, Engineering Assessment and Preliminary Guidelines". Prepared Under Contract No. DTFH61-83-C00101, Federal Highway Administration, FHWA, Washington D. C., (from Frobel, 1991).

Lafleur, J.; Wendling, G.; and Terreault, M. (1986) "Filtration of Broadly Graded Soils by Woven Geotextiles" Proceeding of the Third International Conference on Geotextiles, Vienna, Austria, pp. 657-662

Lafleur, J.; Mlynarek, J.; and Rollin, A. (1989) "Filtration of Broadly Graded Cohesionless Soils", Journal of the Geotechnical Engineering Division, ASCE, Vol. 115, No. 12

Lawson, C. R. (1982) "Filter Criteria for Geotextiles, Relevance and Use" Journal of the Geotechnical Engineering Division, ASCE, GT10, pp. 1300-1317

Lennoz-Gratin, Ch. (1987) "The Use of Geotextiles as Drain Envelopes in France in Connection with Mineral Clogging Risks" Geotextile and Geomembrane, 5, pp. 71-89

Masounave, J.; Denis, R.; and Rollin, A. L. (1980) "Prediction of Hydraulic Properties of Synthetic Non-woven Fabrics used in Geotechnical Work" Canadian Geotechnical Journal, 17, pp. 517-525

Matheron, G. (1971) "Les Polydres Poissoniens Isotropes" Cahier Interne du Centre de Morphologie Mathematique, Fontainebleau, (from Faure, et al 1986^b)

Miller, P. J.; Ho K. W.; and Turnbull, H. R. (1980) "A Study of Filter Fabrics for Geotechnical Applications in New Zeland" Ministry of Works, Central Laboratories, Report No. 2-80/5 (from Fischer, et al 1990)

Mlynarek J.; Lewandowski, B.; Rollin, A. L.; and Blouc, G. (1991) "Soil Geotextile System Interaction" Geotextiles and Geomembranes, 10, pp. 161-176

Moulton, I. K. (1989) "Highway Subdrainage Design", Federal Highway Administration, Report No. FHWA-RD-72-30

Ogink, H. J. M. (1975) "Investigation on the Hydraulic Characteristics of Synthetic Fabrics" Publication No. 146, Delft Hydraulic Laboratories, Delft, The Netherlands

Qureshi, S. (1990) "Long Term Filtration Behavior of Non-woven Geotextiles with Gap Graded and Silty Soils" Msc. Thesis, Syracuse University, Syracuse, N.Y., 173 p.

Prapaharan, S.; and Holtz, R. D. (1987) "Pore Size Distribution of Non-Woven Geotextiles" Report Submitted to Hoeschst-Celanese Corporation, Civil Engineering Department, Purdue University

Prapaharan, S.; Holtz, R. D.; and Luna, J. D. (1989) "Pore Size Distribution of Non-woven Geotextiles" Geotechnical Testing Journal, Vol. 12, No. 4, pp. 261-268.

Rankilor, P. R; (1978) "Designing with Terram" Imperial Chemical Industries report, London

Rankilor P. R. (1981) "Membrane in Ground Engineering" Wiley, 377 p. (from Fischer et al, 1990)

Rigo, J. M.; Lhote, F.; Rollin, A. L.; Mlynarek, J.; and Lombard, G. (1990) "Influence of Geotextile Structure on Pore Size Distribution" Geosynthetics, Microstructure and Performance, ASTM, Special Technical Publication, STP 1076, pp. 90-101

Ringorse-voase, A. J. (1990) "One Dimensional Image Analysis of Soil Structure" Journal of Soil Science, 41, pp. 499-512

Ritter, H. L.; and Drake, L. C. (1945) "Pore Size Distribution in Porous Materials" Journal of Industrial Chemistry, the Analytical Edition, Vol 17, pp. 782-786

Rollin, A. L.; Denis, R.; Estaque, L.; and Masounave, J. (1982) "Hydraulic Behavior of Synthetic Nonwoven Filter Fabrics" Canadian Journal of Chemical Engineering Vol 60, pp. 226-234

Rollin, A. L.; and Lombard, G. (1988) "Mechanisms Affecting Long Term Behavior of Geotextiles" Geotextiles and Geomembranes 7, pp. 119-145

Rollin A. L.; Mlynarek J. R.; and Vidovic A. (1985) "Synthetic Envelope Materials for Subsurface Drainage Tube" Paper Presented at CPTA Annual Meeting, Fort Lauderdale, FL (from Qureshi, 1990)

Rosen, W. J.; and Marks, B. D. (1975) "Investigation of the Filtration Characteristics of a Nonwoven Fabric Filter" Transportation Research Record, TRR 532

Sherard, J. L.; Dunnigan, L. P.; and Talbot, J. R. (1984) "Basic Properties of Sand and Gravel Filters" Journal of the Geotechnical Engineering Division, ASCE, Vol. 110, No. 6, pp. 684-699

Shober, W.; and Teindl, H. (1979) "Filter Criteria for Geotextiles" Proceeding of the Seventh European Conference on Soil Mechanics and Foundation Engineering, Brighton, U. K., pp. 121.

Silveira, A. (1965) "An Analysis of the Problem of Washing Through in Protective Filters" Proceeding of the Sixth International Conference on Soil Mechanics and Foundation Engineering, ICSMFE, Montreal, Canada, 6/27.

Sotton, M.; Leclercq, B.; Fedoroff, N.; Fayoux, D.; and Paute, J. C. (1982) "Contribution to the Study of Clogging of Geotextiles. Morphological Approach" Proceeding of the Second International Conference on Geotextiles, pp. 109-114

Sweetland D. B. (1977) "The Performance of Nonwoven Fabrics as Drainage Screens in Subdrains" Master Thesis, University of Strathclyde, U.K., (from Fischer et al, 1990)

Terzaghi, K.; and Peck, R. B. (1967) Soil Mechanics in Engineering Practice, second edition, John Wiley and Sons

van der Sluys, L.; and Dierickx, W. (1990) "Comparative Study of Different Porometry Determination Methods for Geotextiles" Geotextiles and Geomembranes 9, pp. 183-198

Washburn, E. W. (1921) "A note on Method of Determining the Distribution of Pore Sizes in a Porous Material" Proceedings of National Academy of Science, Vol 7, pp. 115-116

Williams, N. D.; and Houlihan, M. F. (1987) "Evaluation of Interface Friction Properties Between Geosynthetics and Soils" Proceedings of Geosynthetics 87, New Orleans, USA, pp. 616-627.

Williams, N. D.; and Luettich, S. M. (1990) "Laboratory Measurement of Geotextile Filtration Characteristics" Proceeding of the Fourth International Conference on Geotextiles, pp. 273-278

Winslow, D. N.; (1969) "Advance in Experimental techniques for Mercury Intrusion Porosimetry" Journal of Surface and Colloid Science, Vol. 13, Plenum Press, N.Y

Wittmann, L. (1979) "The Process of Soil Filtration its Physics and Approach in Engineering Practice" Proceedings of the Seventh European Conference on Soil Mechanics and Foundation Engineering, Brighton, U. K. pp. 303-310

APPENDICES

Appendix A

AUTOPORE II 9220 V1.07

PAGE 1

SAMPLE NUMBER: 51 OPERATOR: ahmed
 SAMPLE ID: h1114a
 SUBMITTER: ahmed

LP 10:58:44 06/11/91

REP 11:14:12 06/11/91

PENETROMETER NUMBER: 400 ADVANCING CONTACT ANGLE: 135.0 deg
 PENETROMETER CONSTANT: 27.32 $\mu\text{L}/\text{dF}$ RECEDING CONTACT ANGLE: 130.0 deg
 PENETROMETER WEIGHT: 66.2768 g MERCURY SURFACE TENSION: 480.0 dyn/cm
 STEM VOLUME: 1.8360 mL MERCURY DENSITY: 13.5336 g/mL
 MAXIMUM HEAD PRESSURE: 4.4300 psia SAMPLE WEIGHT: 0.0605 g
 PENETROMETER VOLUME: 7.5090 mL SAMPLE+PEN+Hg WEIGHT: 1.0000 g

LOW PRESSURE:

EVACUATION PRESSURE: 15 μmHg
 EVACUATION TIME: 15 min
 MERCURY FILLING PRESSURE: 0.4634 psia
 EQUILIBRATION TIME: 0 sec

INTRUSION DATA SUMMARY

TOTAL INTRUSION VOLUME = 9.5005 mL/g
 TOTAL PORE AREA = 0.275 $\text{sq-m}/\text{g}$
 MEDIAN PORE DIAMETER (VOLUME) = 137.3560 μm
 MEDIAN PORE DIAMETER (AREA) = 129.9010 μm
 AVERAGE PORE DIAMETER (4V/A) = 133.3663 μm
 STEM VOLUME USED = 31 %

AUTOPORE II 9220 V1.07

PAGE 2

SAMPLE NUMBER: 51 OPERATOR: ahmed
 SAMPLE ID: h1114a
 SUBMITTER: ahmed

LP 10:58:44 06/11/91

REP 11:14:12 06/11/91

PRESSURE psia	PORE DIAMETER μm	MEAN DIAMETER μm	CUMULATIVE VOLUME mL/g	INCREMENTAL VOLUME mL/g	DIFFEREN. VOL dV/dD mL/g- μm
0.49	403.1735	403.1735	0.0000	0.0000	0.000E+00
0.57	347.2039	375.1912	0.0335	0.0335	5.990E-04
0.75	263.0145	305.1117	0.3637	0.3352	3.981E-03
0.86	223.6655	245.8400	0.5727	0.2039	5.937E-03
0.94	209.1543	216.9102	0.8129	0.2402	1.231E-02
0.98	199.9425	204.5427	1.1062	0.2933	3.134E-02
1.03	190.3179	195.1302	1.3297	0.2235	2.322E-02
1.07	184.3079	187.3129	1.6565	0.3268	5.435E-02
1.09	180.5077	182.4073	2.0141	0.3576	9.409E-02

1.11	178.8082	178.5570	2.2343	0.2207	5.558E-02
1.19	188.0769	171.3416	2.5001	0.2654	2.520E-02
1.20	183.4196	164.7483	2.3323	0.3327	1.440E-01
1.21	162.9850	163.2023	3.4499	0.5671	1.305E+00
1.21	148.3570	155.6710	3.4499	0.0000	0.000E+00
1.34	146.9601	147.6536	2.9946	0.5447	3.899E-01
1.35	145.5638	146.2620	4.2349	0.2402	1.721E-01
1.39	141.2036	143.3837	4.4416	0.2067	4.741E-02
1.43	137.3631	139.5359	4.6953	0.2542	7.621E-02
1.46	134.5387	136.2034	5.0617	0.3659	1.099E-01
1.51	130.3055	132.6721	5.4779	0.4162	1.115E-01
1.52	129.6981	130.2518	5.3215	0.3436	3.103E-01
1.53	129.6094	129.1537	6.2092	0.3383	3.566E-01
1.53	129.6094	129.6094	6.4612	0.2514	0.000E+00
1.60	122.9335	125.7715	6.7741	0.3129	5.512E-02
1.61	122.3201	122.6263	7.2373	0.4657	7.553E-01
1.62	121.7123	122.0164	7.7965	0.3587	9.199E-01
1.65	121.4715	121.5921	8.0619	0.2654	1.100E+00
1.72	114.6536	113.0626	8.4110	0.3492	5.123E-02
1.76	112.0336	113.2436	8.6983	0.2977	1.098E-01
1.83	107.5129	109.7733	8.8303	0.1816	4.017E-02
1.87	100.1346	103.8233	9.0815	0.2011	2.724E-02
2.26	87.0438	93.5917	9.2536	0.1372	1.430E-02
3.27	50.8990	63.9739	9.4362	0.1676	4.536E-03
13.98	9.5644	30.3217	9.5005	0.0642	1.566E-03

AUTOPORE II 9220 V1.07

PAGE 1

SAMPLE NUMBER: 52 OPERATOR: ahmed
 SAMPLE ID: h1114b
 SUBMITTER: ahmed

LP 10:53:44 06/11/91

REP 11:15:25 06/11/91

PENETROMETER NUMBER: 398 ADVANCING CONTACT ANGLE: 135.0 deg
 PENETROMETER CONSTANT: 27.82 $\mu\text{L}/\text{pF}$ RECEDING CONTACT ANGLE: 130.0 deg
 PENETROMETER WEIGHT: 66.8560 g MERCURY SURFACE TENSION: 480.0 dyn/cm
 STEM VOLUME: 1.8360 mL MERCURY DENSITY: 13.5339 g/mL
 MAXIMUM HEAD PRESSURE: 4.4500 psia SAMPLE WEIGHT: 0.0562 g
 PENETROMETER VOLUME: 7.4460 mL SAMPLE+PEN+Hg WEIGHT: 1.0000 g

LOW PRESSURE:

EVACUATION PRESSURE: 15 μmHg
 EVACUATION TIME: 15 min
 MERCURY FILLING PRESSURE: 0.4884 psia
 EQUILIBRATION TIME: 0 sec

INTRUSION DATA SUMMARY

TOTAL INTRUSION VOLUME = 8.8819 mL/g
 TOTAL PORE AREA = 0.265 sq-m/g
 MEDIAN PORE DIAMETER (VOLUME) = 135.2084 μm
 MEDIAN PORE DIAMETER (AREA) = 129.6094 μm
 AVERAGE PORE DIAMETER (4V/A) = 134.1615 μm
 STEM VOLUME USED = 27 %

AUTOPORE II 9220 V1.07

PAGE 2

SAMPLE NUMBER: 52 OPERATOR: ahmed
 SAMPLE ID: h1114b
 SUBMITTER: ahmed

LP 10:53:44 06/11/91

REP 11:15:25 06/11/91

PRESSURE psia	PORE DIAMETER μm	MEAN DIAMETER μm	CUMULATIVE VOLUME mL/g	INCREMENTAL VOLUME mL/g	DIFFEREN. VOL dV/dD mL/g- μm
0.49	403.1735	403.1735	0.0000	0.0000	0.000E+00
0.57	347.2089	375.1912	0.0363	0.0363	6.480E-04
0.75	263.0145	305.1117	0.3747	0.3385	4.020E-03
0.86	228.6655	245.8400	0.4624	0.0876	2.551E-03
0.94	209.1548	213.9102	0.3976	0.4352	2.230E-02
0.98	199.9425	204.5487	1.0830	0.1904	2.067E-02
1.03	190.3179	195.1202	1.2965	0.2085	2.167E-02
1.07	184.3079	187.3129	1.4869	0.1904	3.168E-02
1.09	180.5077	182.4073	1.6924	0.2055	5.408E-02
1.11	176.6062	178.5570	1.9734	0.2811	7.204E-02
1.13	166.0769	171.3416	2.3482	0.3747	3.559E-02
1.20	163.4196	164.7483	2.7078	0.3596	1.353E-01
1.21	162.9850	163.2023	2.9616	0.2539	5.641E-01
1.21	148.3570	155.6710	2.9616	0.0000	0.000E+00
1.24	146.9601	147.5586	3.4663	0.5047	3.613E-01

1.35	145.5638	146.2620	3.5507	0.1243	1.320E-01
1.39	141.2036	143.3637	3.9106	0.2599	5.961E-02
1.43	137.3631	139.5359	4.1735	0.2629	7.382E-02
1.46	134.5337	136.2034	4.5089	0.3353	1.008E-01
1.51	130.8055	132.6721	4.8504	0.3415	9.148E-02
1.52	129.6981	130.2513	5.1224	0.2720	2.456E-01
1.53	128.6094	129.1537	5.3254	0.2929	2.415E-01
1.53	128.6094	128.6094	5.5516	0.1662	0.000E+00
1.60	122.9625	125.7715	5.8024	0.2508	4.419E-02
1.61	122.3201	122.6263	6.2346	0.4322	7.045E-01
1.62	121.7123	122.0164	6.7302	0.4956	8.161E-01
1.62	121.4715	121.5921	6.9562	0.2267	9.395E-01
1.72	114.6536	113.0625	7.5220	0.5631	3.299E-02
1.76	112.0336	113.3436	7.8604	0.3335	1.292E-01
1.83	107.5129	109.7733	8.1052	0.2448	5.415E-02
1.97	100.1348	103.8258	8.3440	0.2337	3.236E-02
2.25	37.0458	93.5917	8.3857	0.2413	1.648E-02
3.37	50.8990	63.9739	8.7975	0.2115	5.652E-03
19.96	9.3644	30.3617	8.8619	0.0846	2.062E-03

Appendix B

Figure B1. SEM photograph of TR1114 (X15)

Figure B2. SEM photograph of TS600 (X20)

Figure B3. SEM photograph of Ph9 (X15)

Figure B4. SEM photograph of T3341 (X18)

Figure B5. Enlarged photographic print of the thin section of TS700, (X25)

Figure B6. Enlarged photographic print of the thin section of TS800, (X25)

Figure B7. Enlarged photographic print of the thin section of TS1000, (X16)

Figure B8. Enlarged photographic print of the thin section of
Ph4, (X25)

Figure B9. Enlarged photographic print of the thin section of
Ph9, (X25)

Figure B10. Enlarged photographic print of the thin section of
Phl2, (X14)

Figure B11. Enlarged photographic print of the thin section of TR1114, (X25)

Figure B12. Enlarged photographic print of the thin section of TR1125, (X25)

Figure B13. Enlarged photographic print of the thin section of
TR1145, (X17.5)

Figure B14. Enlarged photographic print of the thin section of
A4545, (X25)

Figure B15. Enlarged photographic print of the thin section of A4553, (X25)

PHOTOGRAPH NUMBERS

All photographs are obtainable from Medical Illustration Center of the Veterinary School, Purdue University.

Figure	Number
B5	SB91.348 (4)
B6	SB91.355 (37)
B7	SB91.348 (1)
B8	SB91.355 (35)
B9	SB91.355 (36)
B10	SB91.348 (5)
B11	SB91.355 (32)
B12	SB91.355 (33)
B13	SB91.355 (34)
B14	SB91.348 (7)
B15	SB91.355 (29)
4.7	SB91.355 (30)

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